VOLUME 84 NO. WW1
JANUARY 1958
PART 1

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

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CONTENTS

January, 1958

Papers

	Number
New Retractable Marine Fender System by Palmer W. Roberts and Virgil Blancato	1513
The Columbia River Controlled by Louis H. Foote	1514
Hydraulic Problem Solution on Electronic Computers by Edward A. Lawler and Frank U. Druml	1515
Closure of the Breach in Bayocean Peninsula, Oregon by Harlan E. Brown, Jr., Gerald D. Clark, and Robert J. Pope	1516
Flood control in New England Alden K. Sibley	1517
St. Lawrence Seaway, 27-ft Canals and Channels by William Grothaus and D. M. Ripley	1518
Discussion	1523

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Paper 1513 January, 1958 WW 1

Journal of the
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NEW RETRACTABLE MARINE FENDER SYSTEM

Palmer W. Roberts,[1] M. ASCE and Virgil Blancato[2]
(Proc. Paper 1513)

SYNOPSIS

This paper describes the construction and operation of a new timber type retractable fender system for piers, originally installed at the New York Naval Shipyard. A description of tests performed on this new system for the determination of impact energy absorption capabilities, fender stresses, and reactions within the piers is also included in this paper.

INTRODUCTION

The quest for a marine fender system capable of absorbing the impact energies of berthing vessels, and providing the necessary protection for both the pier and the vessel was begun by marine and harbor designers with the advent of the use of heavy steel vessels. Various systems have been developed employing the principals of gravitation, coil springs, rubber buffers, hydraulic snubbers and pneumatic cylinders. Many of these systems have remained in the trial stages because of excessive initial costs, difficulty of application to existing piers, costly maintenance, insufficient energy absorption qualities, or a combination of any or all of the above reasons. Therefore, the protection of piers and ships has been, for the most part, dependent upon the conventional pile system with its inadequate energy absorption qualities, resultant damage and costly maintenance characteristics.

During the spring of 1954, a 104 foot trial section of a new timber retractable fender system was installed in the New York Naval Shipyard, Brooklyn, New York. As a result of the satisfactory performance of this test section,

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1513 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

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2. Mgr., Structural Branch, Public Works Dept., New York Naval Shipyard, New York, N. Y.

the Bureau of Yards and Docks, United States Navy, authorized the yard-wide replacement of the existing conventional double-wale and pile fender system. At the present writing, 3,500 lineal feet of the new fender system have been installed. The excellent protection afforded, coupled with the low initial and almost negligible maintenance costs, indicate that the retractable fender system meets all of the prerequisites set forth by waterfront and harbor designers.

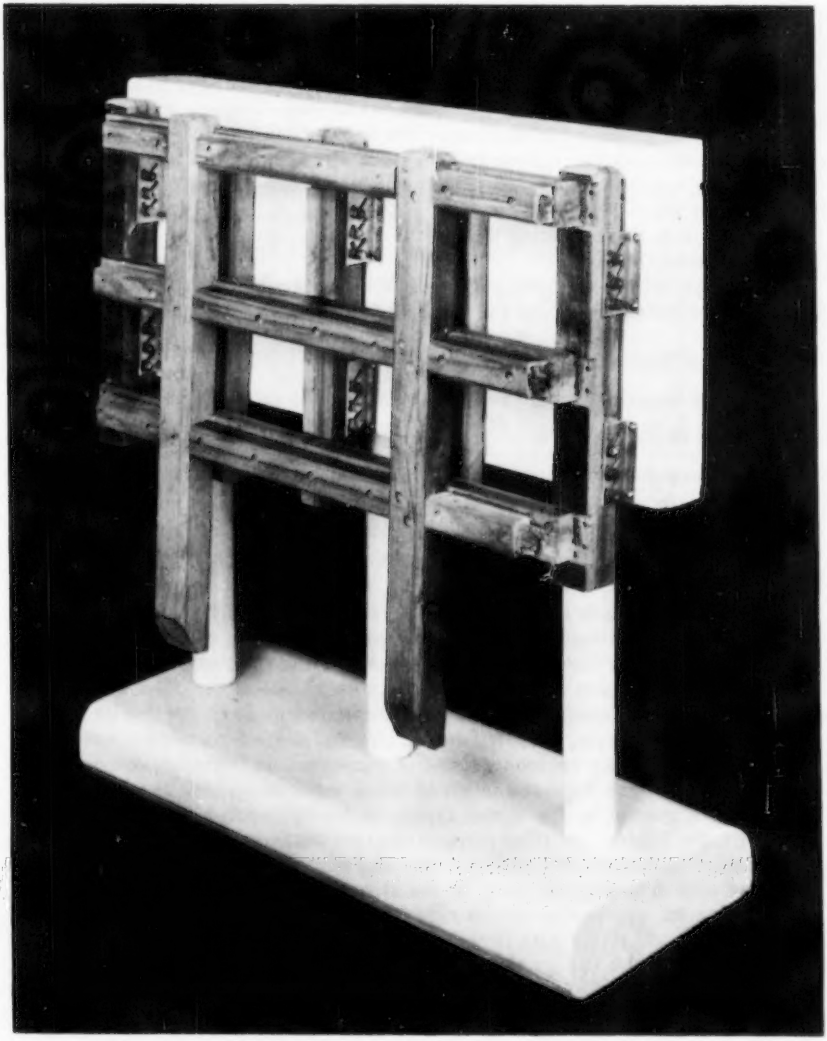
Description

The retractable fender consists of a continuous timber frame suspended by bolts in inclined slotted holes of two rows of U-shaped brackets, spaced 8 to 12 feet apart, fastened to the pier. The frame is composed of vertical contact members at the ship side, bolted at the middle point of three rows of wales, which in turn are fastened to the rear vertical holding posts. Each post is suspended by six 1-1/4 inch diameter bolts inserted in the slots of two brackets; three in each. (Fig. 1) When the retractable fender is subjected to a force greater than the resultant of its weight in the direction of the slots and the friction of the bolts, it moves backward and upward for a distance of 4 or 5 inches depending on the length of the slots in the brackets. The holding posts then bear against the back of the brackets and the wales, after deflecting 1/2 inch, comes to bear against a vertical bearing board fastened to the bulkhead midway between the brackets. Its capacity to absorb large amounts of energy of impact results from the fact that, when a fender is pushed back three or more posts at each side of the contact point intermittently follow the movement, thus gradually increasing the resistance to the acting force. Similarly, when the acting force ceases, the fender returns to its low position with gradually diminishing force due to the intermittent arrival to the lower position of the various displaced posts. In this process the return of the fender to its lower or normal position does not induce in the brackets but a fraction of the impact a gravity fender of similar energy absorption capacity would have created falling four inches. Another outstanding feature of the system is that the posts extending below mean low water are fitted with a third wale near mean low water which gives additional bearing to the fender stub. The upper bracket is provided with V shaped slots to permit the upper part of the fender system, when hit at low level to move upward and outward, while the lower part of the system moves upward and inward. Guided by the slotted holes of the lower brackets, it bears against the vertical bearing boards for added resistance.

Test and Results

Two static pressure tests have been performed on the fender to determine the energy absorption capacity of the system with four inch retraction, and the general behaviour of the various members of the system under stresses. The results obtained from these two tests are as follows:

Test No. 1—One jack applied on one vertical contact fender at point of connection with the middle wale: The jack was bearing on a four legged frame fastened to the pier and pressing the fender inward, toward the pier. It required over 50 tons to push the two adjacent holding posts back to the rear of the bracket and the deflected wales back against the vertical bearing board.



NEW YORK NAVAL SHIPYARD, NAVAL BASE, BROOKLYN 1, N.Y.
NY3-3909 (L)-5-55 23 May 1955
Model of new retractable fender system. Note brackets with slotted holes securing fender to sea wall.

In the backward movement, 3 posts at each side of the jack had been displaced from the lower position at subsequent intervals and with diminishing degree of displacement.

Test No. 2—Three jacks applied on vertical contact fenders spaced 24 feet apart: In this test the jacks were applied in the same manner as described in Test No. 1. The pressure of the jacks was applied simultaneously and the readings of displacement were taken for each 10 tons pressure of each jack.

The load displacement curves shown in Fig. 2 were derived from the results obtained from the two tests.

Curve (1) is drawn by taking the average displacement of the two holding posts adjacent to the jack.

Curve (2) is obtained by using the average displacement of the six posts adjacent to the three jacks and the total load of the jacks at each reading.

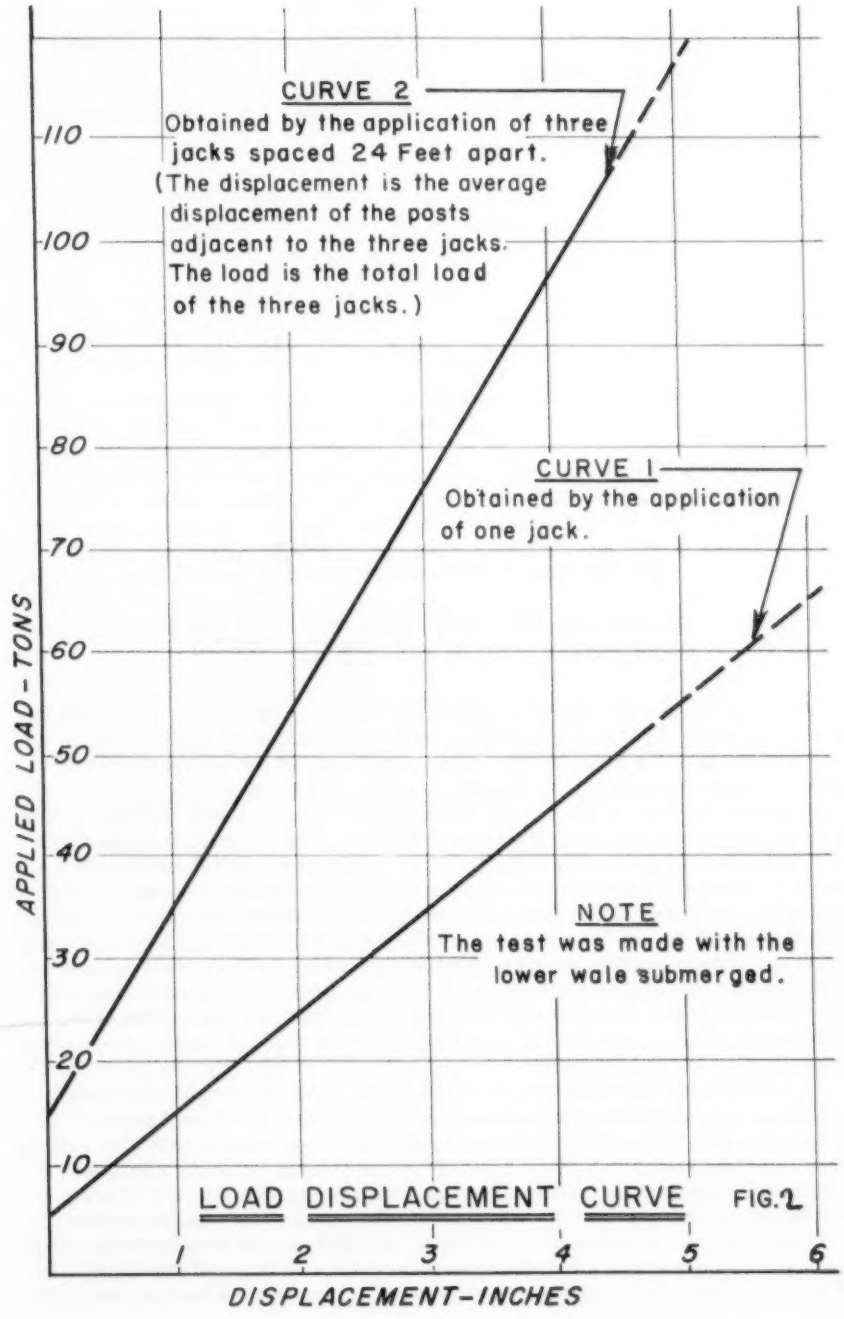
Analysis of the Test—Energy Absorption

The energy absorbed by the retractable fender to move to its back position when one jack is applied is $E = \frac{50 \times 4\text{-}1/2}{2} = 112.5$ inch tons. The energy absorbed by the retractable fender when 3 jacks are applied within 48 feet is $\frac{110 \times 4\text{-}1/2}{2} = 247.5$ inch tons.

Reaction of the Fender

The test with one jack proves that a horizontal force of 50 tons applied on a contact member is required to move the fender back and up to its full retracted position. This force is required to overcome the resistance opposed to the movement by about 24 feet of fender at each side of the point of application of the force. Under this force all the members of the portion of fender involved in the movement, namely, wales, chocks, posts, connecting and suspending bolts, are under various degree of stresses. It is the elastic deformations created by the stresses in all of these members which absorbs most of the energy imparted by the acting force. Actually, when a ship comes in contact with the fender, the first point of contact retracts a certain amount, allowing other members of the fender to make contact at different intervals of time and with different intensity of resistance until the movement of the vessel is stopped, or the fender has reached its full retraction bearing against the pier. The test with three jacks applied to the fender gives an approximate value of the energy the fender is capable of absorbing when the vessel contacts various points within a 48 foot length. It is significant in the property of this fender that due to the flexibility of the frame, when a vessel is breasted against the fender many points of contact with the vessel are established, eliminating the possibility of overstressing any part of the vessel frame.

The load-displacement diagrams drawn from the tests show that the curves are a straight line, indicating that the resistance to the acting force is proportionally increasing with the displacement throughout the retraction. Following the characteristics of the curve it can be deducted that to deflect the fender five inches would require a horizontal force of about 60 tons. It is the opinion of the authors that the 5 or 6 inch retraction system would be required only on exposed piers where large ships would be berthing under the action of strong winds and waves. In this case the dimensions of the wales should be increased in proportion to the acting forces.



LOAD DISPLACEMENT CURVE FIG. 2

Pier Reaction

When a force is applied on a vertical contact member the two adjacent posts will react with one half the value of the acting force. The posts are supported by two brackets, each reacting according to the elevation of the applied force. To simplify the following analysis it is assumed that only one bracket is supporting the post.

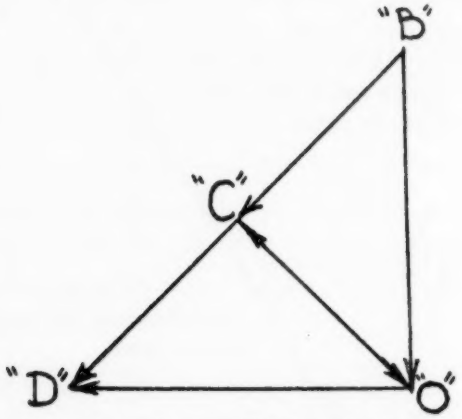


DIAGRAM OF FORCES ACTING ON A BRACKET AT TIME OF RETRACTION.

In the force diagram BO represents the reaction opposed by the fender to the acting force OD. The components of BO perpendicular and parallel to the slot are BC and CO respectively. The components of the acting force OD, perpendicular and parallel to the slot are CD and OC respectively. To obtain equilibrium the forces CO and OC shall be equal and opposite. The two forces BC and CD are equal and in the same direction. Their sum, BD is the force acting against the pier with an inclination, in this case, of 45° with the horizontal. This is the force which creates frictional resistance of the suspending bolts sliding in the bracket slots, increasing considerably the resistance that the gravity alone could oppose to the lifting of the fender. This is a rare case in resilient marine fenders, in which the acting force is used to create a frictional resisting force, increasing in value with the retraction, and employed to spend the acting force. To give an approximate idea of the proportion the friction and gravity contribute in the amount of energy absorption, the following example is cited:

If a 48 foot section of fender, with the lower wale submerged, weighing 8,000 lbs. was suspended by chains of sufficient length to allow 4 inch lift of the center of gravity of fender by swinging, the energy absorbed by the section due to gravity would be 8,000 x 4 = 32,000 inch pounds. This is a fraction of the energy absorbed by the same section supported on slotted brackets retracting 4 inches as shown by the tests. In the stress diagram the vertical and horizontal components of the force BD are BO and OD respectively. The vertical component has a considerable significance in favor of the stability of the pier, because it creates a moment opposite to the overturning moment

created by the horizontal force. The vertical force is the consequence of the inclination of the slots in the brackets, which, while guiding the fender upward, creates a downward component of the acting force. The effect of this downward resultant is more significant when the problem of stability of open narrow finger piers is considered.

Adequacy

The kinetic energy absorption indicated by the test with 3 points contact can be considered derived from the reaction of 48 feet of fender, which gives the capacity absorption of the fender $\frac{247.5}{48} = 5.1$ inch ton per foot.

The energy of impact imparted to a fender by a vessel of 30,000 tons berthing at speed perpendicular to the pier of 24 feet per minute or .4 feet per second will be $E = 1/2\ MV^2 \times Co = \frac{30.000 \times .4^2 \times 12 \times .40}{2 \times 32.2} = 358$ inch tons.

This energy is less than the amount the fender can absorb when acted upon a length of 70 feet. The value of 0.4 given to the coefficient Co is considered adequate for a vessel of deep draft.

The CVA60, a Forrestal class carrier, of 60,000 ton weight, berthing with the same speed as considered in the previous case will impart to the fender 716 inch tons of kinetic energy. For the berthing of this type of vessel, 2 or 4 camels 72 foot long are used as separators between the vessel and the pier, which permit sufficient length of fender to absorb the impact energy even if the speed of berthing should exceed the 24 feet per minute.

It is important to notice that in case of an accidental berthing, when the kinetic energy is far superior to the 400 inch tons, which is used by pier designer as the normal berthing kinetic energy, the hull of the vessel will be bearing on a large number of wood members, which are structurally connected to absorb a good portion of the excess energy through the elastic fiber deformations, and thus prevent damage to the vessel and to the fender.

Resistance to Longitudinal Berthing Forces

In normal berthing, with the vessel approaching the pier at an angle of about 10 degrees with the pier face, the longitudinal force imparted to the fender is limited to the amount of friction the contact between the vessel and the fender can create. The coefficient of friction is dependent on the type of timber and the amount of oil slick covering the surface of contact. The value of the frictional force is a function of the component perpendicular to the pier of the acting force. The longitudinal force imparted to the fender by a berthing vessel is a small fraction of the perpendicular force the fender is called upon to resist.

The retractable fender is well equipped to resist the longitudinal force due to the fact that the frame is continuous for the length of the pier, and the brackets, being wider than the posts, permit the fender to slide on the suspending bolts 3/4 inch, in each direction. At the end of this lateral movement many posts of the system will be bearing on the side of the brackets, getting the support of numerous elements to resist the acting force. This reasoning explains why the 3,500 feet of retractable fender already installed have not suffered, except for light scratches, in the many berthings performed under difficult conditions.

Advantages

The advantages of the retractable fender system are not limited only to the physical characteristic necessary for the protection of ships and piers, but are extended to the cost of erection and maintenance. The component members of the retractable system can be fabricated to the exact dimensions at the mill, assembled in large sections near the site and lowered in units into the brackets previously installed on the pier. This procedure, possible only with this type of fender, eliminates the time often lost as a result of high tides and inclement weather. Maintenance of the new system is anticipated to be very inexpensive. In more than three years since the first section of the 3,500 feet was installed in the New York Naval Shipyard, not a single member had to be replaced. If by an accidental berthing some of the wales should be broken the replacement of them is a simple and inexpensive matter, since each wale is the length of a bent, and only the broken wale need be replaced. The replacement of a vertical contact member does not need the expensive assembling of pile driving equipment. Regardless of the efficiency of a fender, unless it requires little or no maintenance, it will not be adequate. A fender that requires constant maintenance of mechanism soon will become useless because the maintenance eventually will not be at par with the needs of the mechanism. The retractable fender has no mechanism to be maintained.

Applications

The retractable fender has been applied on both bulkhead and open piers. The spacing of the brackets may vary from 8 to 12 feet. On the open piers the upper brackets are easily applicable on the deck frame; the lower brackets can conveniently be located on structural members secured to the bearing piles at elevation corresponding with the horizontal and diagonal bracing of the piles.

Due to the outstanding feature of the system to distribute the acting force to the various brackets adjacent to the contact fenders under action, only a fraction of this force has to be supported by one bracket, and this force is always limited to the capacity of the wales to carry the load into the posts. In consequence no serious damage can come to the bearing piles supporting the brackets.

In conclusion, the retractable fender system has the following outstanding qualifications, which makes it close to an ideal fender:

1. Can be designed to absorb a large amount of energy of impact.
2. The rate of energy absorbed increases with the retraction, making it suitable for berthing of small or large ships.
3. Distributes the energy of impact to many members of the system and to a large area of the pier making it desirable for open piers of light construction.
4. It is economical to build and to maintain.
5. It is long lasting.

Paper 1514 January, 1958 WW 1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

THE COLUMBIA RIVER CONTROLLED

Louis H. Foote,[1] M. ASCE
(Proc. Paper 1514)

ABSTRACT

Continued planning is under way by the Corps of Engineers for development of the water resources of the Columbia River Basin. These studies will take cognizance of changed conditions and needs since the Corps' comprehensive report of 1948. This paper outlines these changes and the problems being encountered in providing an extended long-range plan for resource development.

What is involved in control of the Columbia River? Much has been said on this subject and much more will be said about it before posterity—many generations in the future—can declare it an accomplished fact. Even then engineering groups will look back on the history of the Columbia River development and philosophize on what should have been done and what could have been done, had engineers in this day known what they will know then.

Engineers of today recognize that as life gets more complicated, so do the problems of planning for the future. The fact must be faced that, in the words of Robert Burns, "the best-laid schemes o' mice an' men gang aft agley." Truer words were never spoken when applied to long-range planning. There can be no final long-range plan; all plans which project very far into the future must be flexible, and engineers who plan for the future must be prepared to adjust for conditions as they change. The only alternative is a perfect insight into the future with a crystal ball that shows everything. Since this is not available, planning becomes an art, rather than a science, and, with the passing of time, engineers must alter their sights and take a new look.

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1514 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

1. Brigadier General, Div. Engr., U. S. Army Engr. Div., North Pacific, Corps of Engrs., Portland, Ore.

Looking back, it was only 25 or 30 years ago that engineers first undertook to lay out a long-range plan for the Columbia River. Many of today's projects were proposed in the first Columbia River "308" Report written in 1930 and 1931, and the 10-dam plan recommended in 1932 by the Board of Engineers for Rivers and Harbors for ultimate development of the main Columbia River is essentially the plan in use today. In 1947 and 1948 a review report was written on comprehensive planning for the Columbia River Basin and further developments were recommended in the light of conditions appraised at that time and another step forward was taken. Today, another appraisal is being made of the resources of the basin, and based on the new findings, the plans of the past will be realigned again for the future.

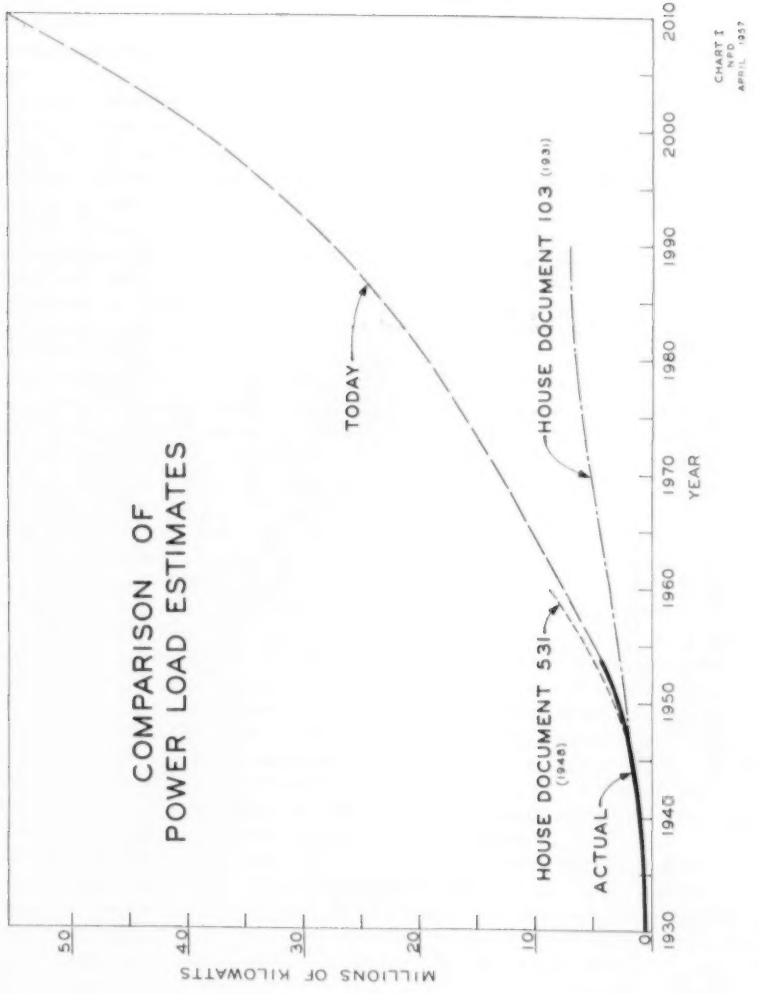
Present concepts of water resource planning for the Columbia are slightly different than they were in the past, and yet the changes have been so gradual in many cases that they are frequently overlooked and engineers continue to analyze problems with the old yardstick. A brief summary of these changes will make them more apparent.

The most important use, moneywise, of water resources is for the generation of hydroelectric power. Even the early studies recognized the tremendous power potential of the Columbia River; however, the earliest estimates of future load were generally conservative. Chart I is a plotting of basin load estimates contained in the 1931 report and the 1948 report, together with the November 1956 estimate prepared jointly by the Federal Power Commission and the Bonneville Power Administration which has been tentatively accepted for the Corps of Engineers' current review report. While there has been a great change in thinking since 1931, the major change since 1948 is just in extension farther into the future. In 1931 there was no question but that there was adequate hydro power potential to meet the future load which was expected to stop growing about 1990. In 1948 the question was again raised but it was considered that the time when hydro generation would run out and thermal power would be required was so far in the future that it would have little bearing on the economics of the projects being recommended at that time. Space for future units was provided for, however, with this eventuality in mind.

Chart II shows a forecast of average annual energy requirements for the Pacific Northwest and illustrates the probable capabilities of obtainable, economic hydro generation and how the future load beyond that amount probably will be met with steam—both fossil-fuel fired and atomic. This situation, which appears inevitable now, requires several changes in the power approach.

In 1948 the basic concept for power analysis was an all-hydro system with only incidental small amounts of the more expensive thermal energy used during breakdowns, extreme critical peaking conditions and similar emergencies. The basic problem was consequently one of providing enough peaking capacity and energy to meet the critical-period requirements. Storage was needed to bridge the critical period. Extra generation obtainable during better than critical water conditions is the interruptible and secondary power available in any hydro system.

Chart III illustrates the present thinking of how the thermal generation will enter the present hydro system. The chart should be considered more illustrative than factual since the data used to plot the curves are very tentative estimates. The lower shaded area represents hydro energy and the shaded area just above it represents thermal energy. The average load


COMPARISON OF
POWER LOAD ESTIMATES
TODAY
HOUSE DOCUMENT 103 (1931)
HOUSE DOCUMENT 531
(1948)
ACTUAL
MILLIONS OF KILOWATTS
0
10
20
30
40
50
1930
1940
1950
1960
1970
1980
1990
2000
2010
YEAR
CHART I
N.P.D.
APRIL 1957

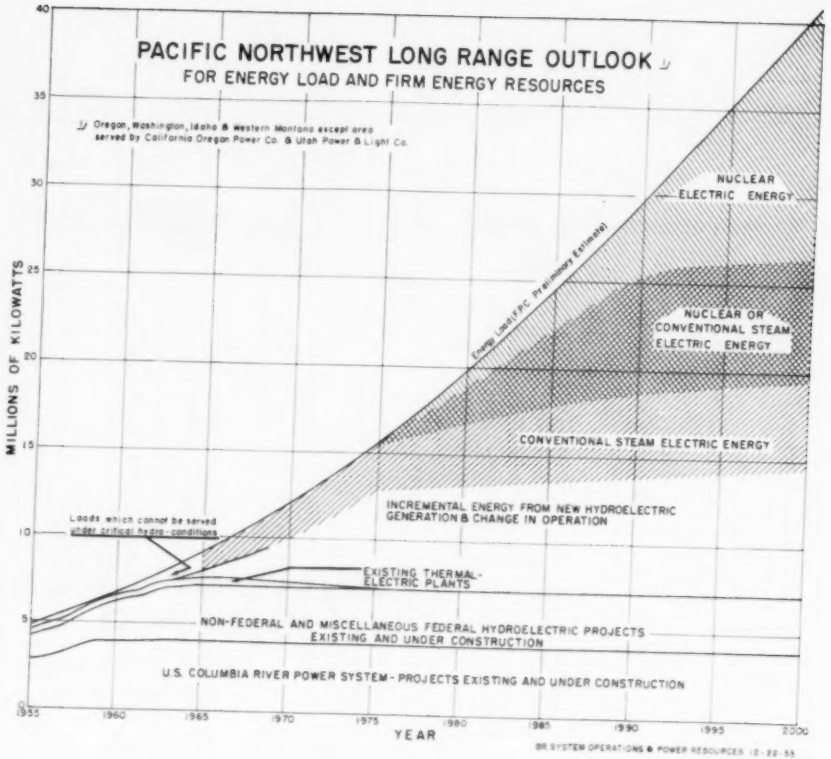

PACIFIC NORTHWEST LONG RANGE OUTLOOK 1/
FOR ENERGY LOAD AND FIRM ENERGY RESOURCES
1/ Oregon, Washington, Idaho & western Montana except area served by California Oregon Power Co. & Utah Power & Light Co.
MILLIONS OF KILOWATTS
YEAR
NUCLEAR ELECTRIC ENERGY
Energy Load (FPC Preliminary Estimate)
NUCLEAR OR CONVENTIONAL STEAM ELECTRIC ENERGY
CONVENTIONAL STEAM ELECTRIC ENERGY
INCREMENTAL ENERGY FROM NEW HYDROELECTRIC GENERATION & CHANGE IN OPERATION
Loads which cannot be served under critical hydro conditions
EXISTING THERMAL-ELECTRIC PLANTS
NON-FEDERAL AND MISCELLANEOUS FEDERAL HYDROELECTRIC PROJECTS EXISTING AND UNDER CONSTRUCTION
U.S. COLUMBIA RIVER POWER SYSTEM - PROJECTS EXISTING AND UNDER CONSTRUCTION
0
5
10
15
20
25
30
35
40
1955
1960
1965
1970
1975
1980
1985
1990
1995
2000
BR SYSTEM OPERATIONS & POWER RESOURCES 12-22-55

CHART II

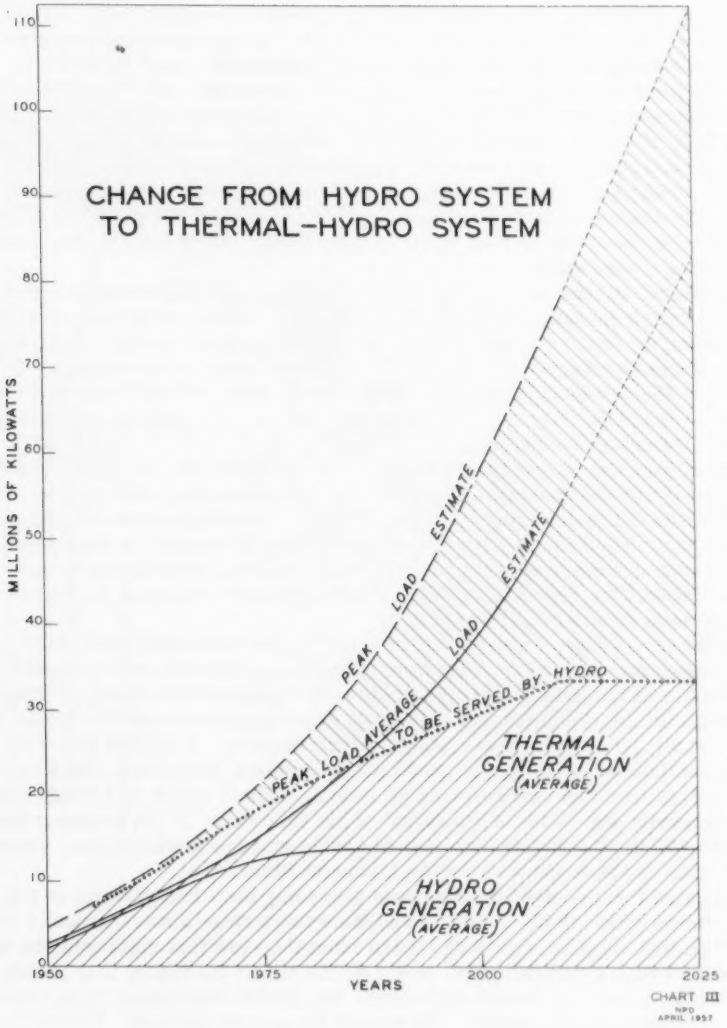
estimate corresponds to the curve of average annual energy shown on the previous chart. The cross-hatched areas represent peaking capacity—hydro and thermal—with the top line representing the estimated system total peak requirement corresponding to the average annual energy forecast.

There are, generally speaking, three periods reflected in this chart. First, the essentially all-hydro period up to sometime after 1975, at which time all of the obtainable economic hydro projects are assumed completed. At this point, increasing amounts of thermal generation must be added to meet the growing energy requirements. Additional units will be added at the hydro plants, however, for capacity, and every effort will be made to utilize every drop of water. Storage will be used as much in the second period as in the first, and possibly more since the load will be so great that it will be economical to use all refillable storage every year for steam replacement energy. Today storage drawdown is frequently stopped after the peak has been met, and the reservoir is then refilled up to flood control limitations, even though it is known water will be spilled later on.

The second period will end sometime after the year 2000 when no more capacity can be installed at hydro projects. This will be fixed by operational limitations, as well as by physical and economic considerations. After this time thermal generation will be required for both capacity and energy. Of course by now the peaking requirements are so great that the cheaper capacity sources must be fully utilized and great care will be taken to insure a full head at all the storage projects every year during the winter season when maximum capacity is required. Storage will be drawn out very judiciously after that time, and its need and use will be somewhat less than before. This latter period is the one frequently referred to as having such an important bearing on the need for and economics of storage for power. It does represent a major change in storage concept but it will still be possible to use a substantial amount, say 20,000,000 to 25,000,000 acre-feet, and it may not come as soon as some have envisioned.

Justification of large amounts of storage for the Columbia Basin to be used only infrequently for power generation purposes is difficult and frequently indefensible in the light of the other resource conflicts that result. Storage is valuable in the Columbia and extremely necessary, to a certain degree, for flood control, irrigation, power and for other reasons. It may be possible to use all obtainable. However, it would not be anything like 150 to 180,000,000 acre-feet, which is the average annual runoff. This is 2-1/2 to 3 times that of the Missouri and 10 to 12 times that of the Colorado. In the neighborhood of 30,000,000 to 40,000,000 acre-feet of usable storage would suffice, which is less than that now apparent on the Missouri River.

Insofar as flood control is concerned there has been little change in the basic concept since 1948. Prior to that time there were several schools of thought on this subject. Opinions varied from the view that reforestation and soil conservation practices alone would do the job to the theory that levees alone would suffice—or reservoirs alone. By 1948 it was generally accepted that a combination of all these methods was the proper concept. The occurrence of major floods in 1948 and in subsequent years has proven that it can still happen here—a fact that wasn't fully appreciated in 1931. Adequate flood control is now accepted as a must for the Columbia River Basin. Much progress has been made since 1948 in the science of forecasting major floods on the Columbia River. The fact that major floods on the Columbia are basically the result of a general rapid spring melting of large accumulations of


CHANGE FROM HYDRO SYSTEM
TO THERMAL-HYDRO SYSTEM
MILLIONS OF KILOWATTS
110
100
90
80
70
60
50
40
30
20
10
0
PEAK LOAD ESTIMATE
PEAK LOAD AVERAGE LOAD ESTIMATE
TO BE SERVED BY HYDRO
THERMAL GENERATION (AVERAGE)
HYDRO GENERATION (AVERAGE)
1950
1975
2000
2025
YEARS
CHART III
NPD
APRIL 1957

winter snows makes the problem of forecasting a little less nebulous than in areas where floods are essentially the result of heavy rains. Because the spring runoff volumes are subject to reasonably good estimates, the use of storage projects for flood control presents only minor conflicts with their use for recreation in the summer and for irrigation and power generation in the later summer, fall and winter, which automatically draws the reservoirs down to provide the storage space to be filled in the spring flood-control operation.

Although the goals that were established in 1948 are valid today, they are, unfortunately, still unattainable. Of the 21,000,000 acre-feet of storage in the Main Control Plan which was found necessary to control the 1894 flood in H. D. 531, the Corps of Engineers' 1948 Report, there is existing today only 5,190,000 acre-feet which can be used, and the present outlook is that only an additional 4,700,000 acre-feet will be achieved in the immediately prospective future.

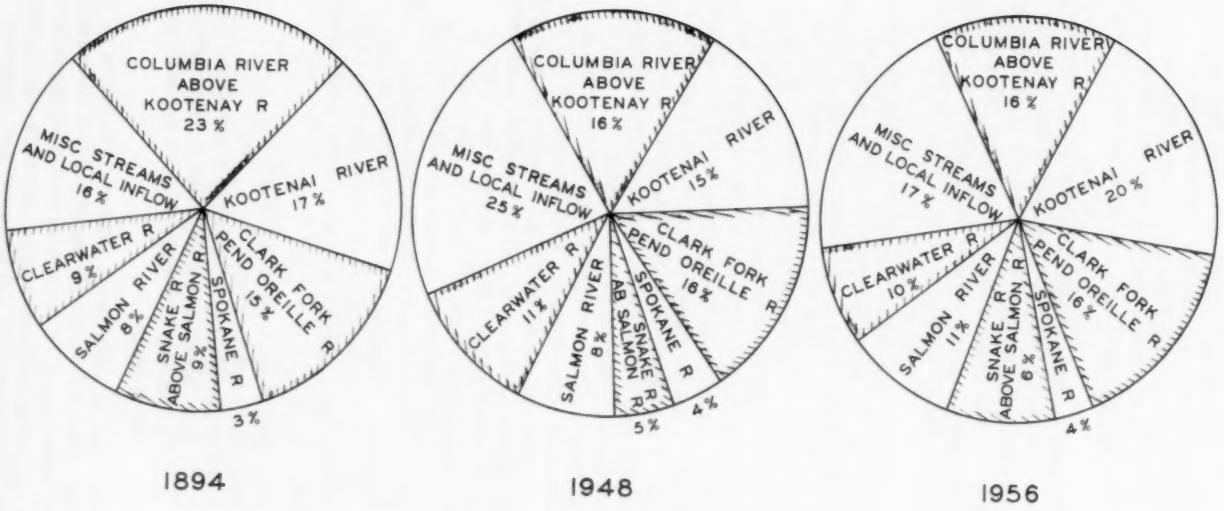
Table 1 shows the flood control storage of the Main Control Plan of H. D. 531 distributed by major basins. It will be noted that the storage available for flood control use totals 27,000,000 acre-feet, in round numbers. Similarly, the amount of storage used in that Main Control Plan for control of the 1894 flood to 800,000 cfs at The Dalles totals 21,000,000 acre-feet, in round numbers. Table 2 shows the present status of the Main Control Plan in storage usable at-site. In addition to what is existing or in reasonably immediate prospect, more than twice as much usable storage is needed to control the 1894 flood to 800,000 cfs. Various things have materialized that have interfered with completion of that Main Control Plan. These are the changed conditions already mentioned. Now it will require a new appraisal and a new Main Control Plan if flood control is to be achieved.

Table 3 shows possible additional storage projects with which to work. Actually the 8-1/2 million acre-feet in the first two groups would be about enough, assuming the depleted flows estimated by 1985. A little more will be needed until that time, and the question arises, will it be possible to achieve all twelve projects listed? It is probable that one or more of the controversial projects shown in the third group—"Others"—will be required.

There has been much discussion of where the storage should be located. Chart IV illustrates the source of the three major floods most familiar to the region—those of 1894, 1948 and 1956. There is a great similarity in the three diagrams. The total "pie" represents the observed volumes in each case passing The Dalles during the flood control period. These total volumes for comparison are as follows:

1894	72,300,000 Acre-feet	- the largest flood of record.
1948	57,800,000 Acre-feet	- the third largest.
1956	41,000,000 Acre-feet	- about Number 11 actually but it could have been Number 4 without the flood control operations that were carried out with existing storage.

Chart V is an "exploded pie" for the 1894 flood. The control period is shown on the little hydrograph. Hatched on the pieces of pie this time are the portions of each contribution which would be controlled by the storages in one of the prospective systems, assuming 1985 conditions. Actually, the system consists of the "present outlook" storage of 9,890,000 acre-feet shown on Table 2, the recommended Clearwater projects of 3,730,000 acre-feet and the


COLUMBIA RIVER ABOVE KOOTENAY R 23 %
KOOTENAI RIVER 17 %
CLARK FORK PEND OREILLE R 15 %
SPOKANE R 3 %
SNAKE R ABOVE SALMON R 9 %
SALMON RIVER 8 %
CLEARWATER R 9 %
MISC STREAMS AND LOCAL INFLOW 16 %
1894
COLUMBIA RIVER ABOVE KOOTENAY R 16 %
KOOTENAI RIVER 15 %
CLARK FORK PEND OREILLE R 16 %
SPOKANE R 4 %
SNAKE R AB SALMON R 5 %
SALMON RIVER 8 %
CLEARWATER R 11 %
MISC STREAMS AND LOCAL INFLOW 25 %
1948
COLUMBIA RIVER ABOVE KOOTENAY R 16 %
KOOTENAI RIVER 20 %
CLARK FORK PEND OREILLE R 16 %
SPOKANE R 4 %
SNAKE R ABOVE SALMON R 6 %
SALMON RIVER 11 %
CLEARWATER R 10 %
MISC STREAMS AND LOCAL INFLOW 17 %
1956

SOURCE OF MAJOR FLOODS

VOLUME CONTRIBUTED DURING
FLOOD CONTROL OPERATIONS

CHART IX
NPD
APRIL 1957

TABLE I

FLOOD CONTROL STORAGE, MAIN CONTROL PLAN
HD 531

Basin	Project	Stream	Total Storage Available for FC Ac. Ft.	Storage used to Control 1894 Flood Ac. Ft.
Kootenai	Libby	Kootenai R.	4,250,000	3,900,000
Clark Fork	Glacier View	NF Flathead	3,160,000	1,800,000
	Hungry Horse	SF Flathead	2,980,000	2,100,000
		Subtotal	6,140,000	3,900,000
Snake	Palisades	Snake	1,200,000	1,200,000
	Cascades	Payette	650,000	160,000
	Garden Valley	Payette	1,250,000	300,000
	Anderson Ranch	Boise	420,000)	
	Arrowrock	Boise	285,000)	230,000
	Lucky Peak	Boise	295,000)	
	Hells Canyon	Snake	3,280,000	2,600,000
		Subtotal	7,380,000	4,490,000
Columbia	Grand Coulee	Columbia	5,120,000	5,100,000
	Priest Rapids	Columbia	2,100,000	2,100,000
	John Day	Columbia	2,000,000	1,400,000
		Subtotal	9,220,000	8,600,000
	Total MCP		26,990,000	20,890,000

TABLE 2

PRESENT STATUS MAIN CONTROL PLAN STORAGE

Project	Flood Control Storage usable at site to control 1894 Flood: Main Control Plan (acre feet)	Present Outlook (acre feet)
Existing		
Hungry Horse	2,100,000	2,100,000
Grand Coulee	1,240,000*	1,500,000 1/
Palisades	1,200,000 2/	1,200,000 2/
Payette & Boise Rivers	390,000	390,000
Subtotal	4,930,000	5,190,000
Additional Proposed in H. D. 531		
Grand Coulee (increase)	3,860,000	2,700,000 4/
Glacier View	1,800,000	— 5/
Libby	3,900,000 6/	— 6/
Priest Rapids	2,100,000	500,000 7/
John Day	1,400,000 3/	500,000 8/
Payette & Boise Rivers	300,000	— 9/
Hells Canyon	2,600,000	1,000,000 10/
Subtotal	15,960,000	4,700,000
Total	20,890,000	9,890,000

* Amount formerly usable without use of outlets. As previously reported.

1/ Fully usable in 1956 and assumed to be available subsequently through use of 38 of 40 outlets in two top tiers but without additional upstream storage.

2/ Operable for storage in 1957.

3/ 2,000,000 acre feet was available for use as required.

4/ With 60 outlets. Full increase to presently estimated capacity of 5,230,000 acre feet not effective until other major storage above Grand Coulee is developed.

5/ Not recommended because of objections by recreation and wildlife interests.

6/ Authorized but construction delayed pending completion of negotiations with Canada. Current plan (Mile 217) provides for 5,010,000 AF of usable storage.

7/ Alternate 2-dam plan of Grant County PUD #2 would provide only 500,000 AF.

8/ Authorized but because of objections to surcharge storage, recommended modification to provide 500,000 AF submitted to Congress.

9/ Recommended Garden Valley project, Payette River, not authorized.

10/ Not authorized. Alternate Brownlee project of Idaho Power Company would provide 1,000,000 AF. Pending Pleasant Valley Project would provide additional 500,000 AF if constructed with adequate outlet capacity.

TABLE 3

POSSIBLE ADDITIONAL FLOOD CONTROL STORAGE
(based on studies to April 1957 - Preliminary data)

PROJECT	STREAM	Total Storage Available (acre feet)	Usable for control 1894 flood to 800,000 cfs at The Dalles (acre feet)
PREVIOUSLY RECOMMENDED			
Bruces Eddy	NF Clearwater	1,430,000	1,430,000
Penny Cliffs	MF Clearwater	2,300,000	2,300,000
TOTAL		3,730,000	3,730,000
UNDER ACTIVE STUDY			
Long Meadows	Yaak River	400,000	400,000
Spruce Park	MF Flathead	300,000	300,000
Flathead Lake Outlet	Flathead	500,000	500,000
Buffalo Rapids	Flathead	670,000	670,000
Ninemile Prairie	Blackfoot	960,000	720,000
Enaville	Coeur d'Alene	700,000	600,000
Wenaha	Grande Ronde	1,000,000	1,000,000
Garden Valley	Payette	1,250,000	600,000 1/
Pleasant Valley	Snake	500,000	500,000
TOTAL		6,280,000	5,290,000
OTHERS (Assumed in alternate systems)			
Chiwawa	Wenatchee	150,000	150,000
Paradise	Clark Fork	4,080,000	4,080,000
Libby	Kootenai	5,010,000 2/	5,010,000 2/
Mica Creek	Columbia in Canada	11,700,000	7,670,000
Arrow Lakes	Columbia in Canada	3,030,000	1,200,000
Smoky Range	NF Flathead	1,500,000	1,500,000
Nez Perce (1490)	Snake	4,120,000	4,120,000
High Mountain Sheep(1490)	Snake	1,600,000	1,300,000

1/ Garden Valley shown operated for downstream flood control. Only 300,000 acre-feet used in H. D. 531 operated for local flood control with incidental downstream benefits. Similar operation will also increase usable storages at Cascade Reservoir from 160,000 acre-feet to 300,000 acre-feet. Additional 30,000 acre-feet assumed used at Deadwood Reservoir.

2/ Libby data based on project at Mile 217.0. Modified project since H. D. 531.

TABLE 4

COLUMBIA RIVER BASIN

SUMMARY OF RESERVOIR REGULATION FOR FLOOD CONTROL
1894 FLOOD

	Storage Aval. 1,000,000 AF 1/	Storage Used 1,000,000 AF 2/	Max. Discharge at The Dalles (cfs)	Maximum Stage at Vancouver (feet)	Total Stage Reduction (feet)
Unregulated, 1894 Flood	-	-	1,240,000	34.4	-
Main Control Plan (HD 531)	27.0	20.9	800,000	26.6	7.8
Existing storage and under construction (as of Apr 57)	8.5	6.4	1,050,000	31.3	3.1
Study Plan A	22.2	19.2	800,000	26.6	7.8
Study Plan B	28.4	25.3	690,000	24.0	10.4
Study Plan C	34.5	31.6	610,000	22.0	12.4
Study Plan D	49.2	38.9	600,000	21.7	12.7

1/ Total usable storage at projects operated for flood control benefit.

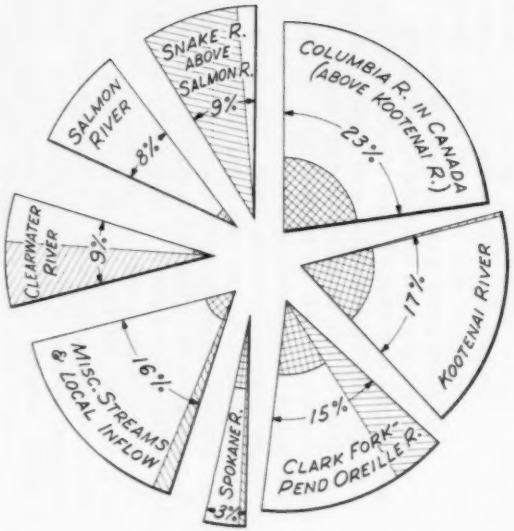
2/ Total volume of water stored at Flood Control reservoirs during entire flood period, including "factor of safety" storage.

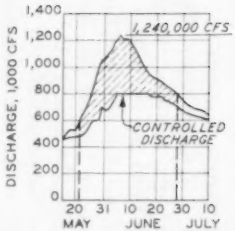
5,290,000 acre-feet of storage under active study, shown on Table 3. The system thus totals 18,910,000 acre-feet of at-site storage usable for control of this flood. To this was added an equivalent of about 2,000,000 acre-feet to represent the irrigation depletions now forecast for about 1985.

Table 4 shows the effect of various amounts of storage on the 1894 flood at The Dalles. It will be noted that the storage existing and under construction, amounting to 6,400,000 acre-feet usable, will reduce the 1894 flood from 1,240,000 cfs to 1,050,000 cfs at The Dalles. The larger systems include Canadian storage in addition to various amounts of U. S. storage. Chart VI shows graphically essentially the same information, with, however, marks on the right-hand margin to indicate the major damage level and the minor damage level. Below about 500,000 cfs there is little flood damage except to marginal farm lands and a few waterfront facilities. Above 730,000 cfs, however, which is 25 feet on the Vancouver, Washington gage, the damage increases quite rapidly. That explains in part the reason for selection of 800.000 cfs as the initial goal in H. D. 531. Control below that amount would produce further benefits but while the Main Control Plan with levees and storage as envisioned in H. D. 531 could be credited with flood control benefits of approximately $35,000,000, complete control by storage below 800,000 cfs would net less than 10 percent more in the way of flood control benefits and these would be largely due to land enhancement rather than damage prevention.

Navigation on the Columbia River still continues to grow. Chart VII plots the history of navigation on the Columbia River, through The Dalles-Celilo Canal section, since 1874. Completion of The Dalles-Celilo Canal in 1915 started the first upswing in tonnage but this later fell off until Bonneville was completed in 1938 to provide slack water past the Cascade Rapids section of the river. Since that time river tonnages have greatly increased. A further upswing may be expected with completion of slack water above The Dalles lock which now replaces the old Dalles-Celilo Canal. After completion of the John Day project, slack water will be available to river navigation from Bonneville to the Pasco-Kennewick area on McNary pool. Ice Harbor Dam is now under construction and in the foreseeable future slack-water navigation is anticipated up the Snake River to Lewiston, Idaho. The navigation concept hasn't changed since 1948 although the new look ahead invites consideration of navigation on up the Columbia River to Wenatchee, Washington, and possibly beyond. Also, the possibility of further low head dams on the Snake River above Lewiston indicates that slack water may be extended on upstream to replace open river improvement previously recommended. In 1948 the tonnage forecast was about 5,000,000 tons for this stretch of the river. This was the estimated 50-year average, following a development period after completion of the navigation portion of the Main Control Plan. The current estimate is about 7,500,000 tons for a similar 50-year average but for a slightly later period starting in 1975. This compares with slightly over 1,000,000 tons passed in 1955 or an estimated increase of about 700 percent in the next 50 years. This, considering an increase of over 1000 percent in the last 20 years, does not seem quite as fantastic.

Navigation isn't without its problems, however. The greater the peaking operation required at the run-of-river power plants, the greater the navigation difficulty, particularly through narrow stretches in the upper reservoir areas and at downstream lock approaches. This conflict has been evidenced in operation of McNary where variable flows resulting from load





1894 FLOOD HYDROGRAPH AT THE DALLES

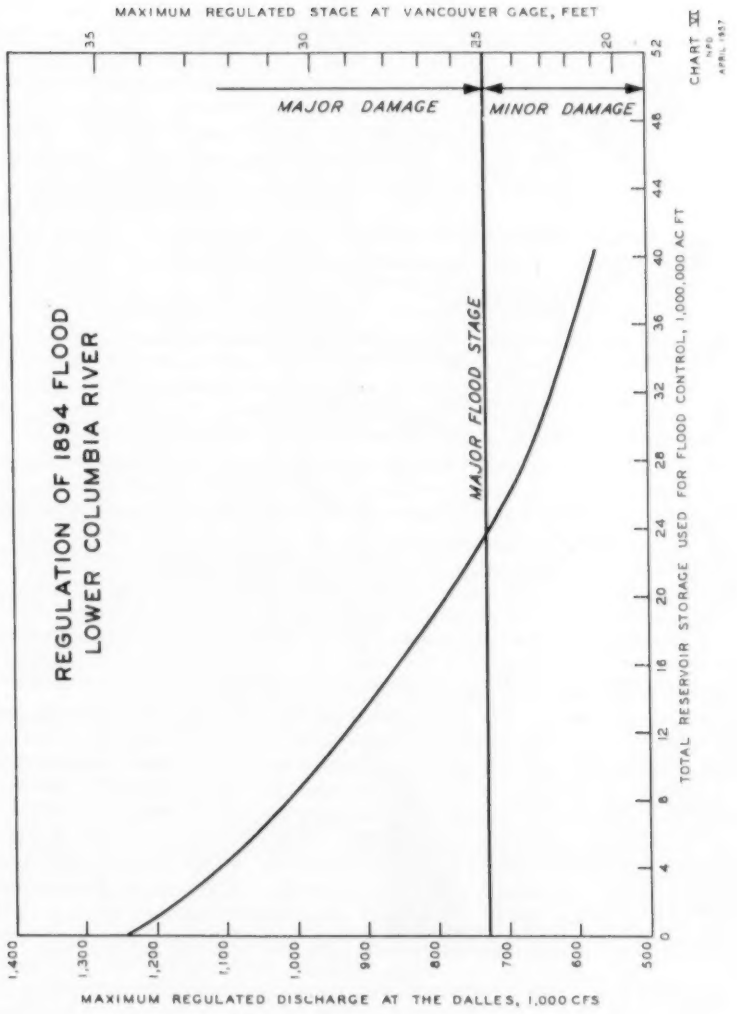
CONTROL AFFORDED BY EXISTING AND POTENTIAL RESERVOIRS WITHIN TRIBUTARY AREA

DOWNSTREAM CONTROL AFFORDED BY EXISTING AND POTENTIAL MAIN STEM COLUMBIA RIVER PROJECTS

1894 FLOOD

DIAGRAM SHOWS PORTION OF FLOOD VOLUME CONTRIBUTED BY VARIOUS TRIBUTARIES DURING THE PERIOD OF FLOOD CONTROL OPERATIONS IN THE INTEREST OF THE DALLES AND DOWNSTREAM.

CHART IX
NPD
APRIL 1951


MAXIMUM REGULATED STAGE AT VANCOUVER GAGE, FEET
35
30
25
20
52
MAJOR DAMAGE
MINOR DAMAGE
48
44
40
36
32
28
24
20
18
12
8
4
0
TOTAL RESERVOIR STORAGE USED FOR FLOOD CONTROL, 1,000,000 AC FT
CHART XI
REGULATION OF 1894 FLOOD
LOWER COLUMBIA RIVER
MAJOR FLOOD STAGE
1,400
1,300
1,200
1,100
1,000
900
800
700
600
500
MAXIMUM REGULATED DISCHARGE AT THE DALLES, 1,000 CFS

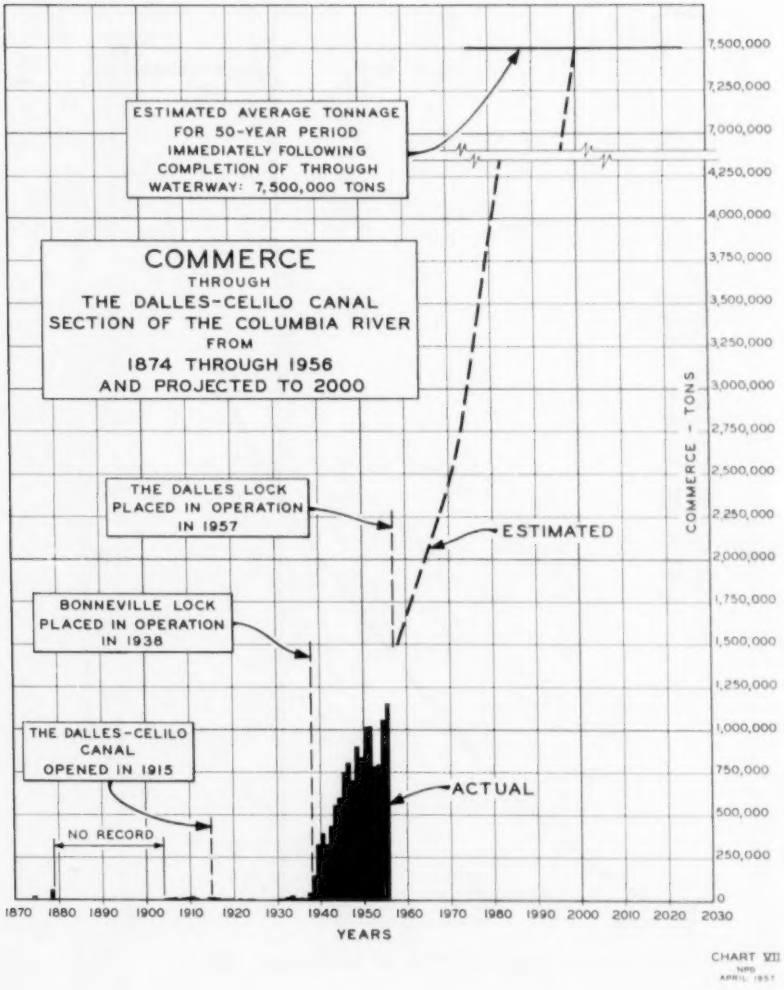


CHART VII
NPD
APRIL 1957

factored operation has interfered with navigation in the open river stretch of the Columbia, which remains until John Day is completed. This problem presents a possible controlling factor in power operations or in navigation growth which must be given serious attention in planning today.

Irrigation developments throughout the basin are continuing and the increased water use for irrigation is having a marked influence in many ways. In 1948, a careful study was made of future irrigation potentials with a view to taking into account in all power studies the effect of future upstream depletions. Future depletions were estimated liberally in order to be conservative in the estimate of future power benefits from downstream projects. That was not necessarily a new concept in 1948 but it was not a factor commonly applied before. Of course now the horizon has been extended further into the future with the result that today the estimated future depletions are greater for the projects now being considered.

In the matter of recreation there has been a tremendous upswing of interest since 1948. This is being evidenced not only locally, in connection with individual projects, but also at the Congressional level where legislation has been under consideration to establish a national policy with respect to recreation. In 1948 recreation needs and values were considered but there was a lack of local interest in this area where there are so many recreational opportunities. Today, however, there is a serious interest not only in the preservation of present recreational opportunities but also in developing new opportunities. This interest not only continues strong with sportsmen and conservation groups but is now actually violent when the infringement of man-made dams and reservoirs on natural conditions is involved. As more and more leisure time becomes available for recreational activities and the need for increased development of water resources becomes greater, the conflict becomes critical. Comprehensive studies now being conducted include very serious attention to the matter of recreation. The National Park Service has agreed to undertake a recreational study in connection with each project considered and the Corps of Engineers' new report will be comprehensive in this field also.

The pressure of civilization is forcing the development of water resources farther and farther upstream and wild, natural areas are now threatened by dams and reservoirs. Facing the problem of providing more facilities for greater recreational use on one hand, it is found that here increased use is not desired. The sportsman, hunter and fisherman not only oppose any threat to the wildlife resource but also join the conservationist who is dedicated to preserving certain areas in a natural primitive condition. Together they present a formidable factor because there is no remedial measure possible except to build no dam at all. Their position is respected, and every effort is made to avoid true wilderness areas. However, if every undeveloped stream were to be included in that category and preserved, the list of obtainable projects must be reduced even further.

As to the fishery resource, in addition to a $5,000,000 sports fishery, there is a $10,000,000 to $15,000,000 commercial salmon fishery associated with the Columbia River which is very important to the region and to the nation. Both the steelhead and the salmon are anadromous fish which in their life cycle pass downstream from their upstream river, stream or lake bed spawning ground to the ocean, and then later as adult fish, find their way back upstream to the place of their origin to spawn and start a new cycle. The Columbia River and its tributaries support a major proportion of the remain-

ing national wealth in this resource. The problem of anadromous fish and dams has existed on the Columbia River for many years. It has long been recognized but not much was done to reconcile the conflict until quite recently. In an effort to clear up some of the many questions that are outstanding, the Corps of Engineers, in collaboration with Federal and State fish and game agencies, initiated a program of engineering research in 1953 which is now beginning to provide some answers.

Specifically, the Corps of Engineers' Fisheries Engineering Research program was designed to find less expensive and better means of passing fish both upstream and downstream over dams. Although the fish-passage facilities at Bonneville and McNary are proving generally satisfactory, research indicates that the ladders could be made steeper and narrower and still be adequate. In addition to economies in construction cost, there can be a saving in the amount of water which would bypass the turbines; hence, a saving in operating costs at existing as well as future projects. Research also offers prospects of finding means of guiding downstream migrants away from spillway and turbine hazards and into facilities for handling the fingerlings safely downstream. Another prospective achievement is the proving out of artificial spawning grounds for salmon and steelhead. If artificial spawning areas can be constructed to effectively and economically provide replacement for natural spawning areas flooded by dams, another obstacle will be cleared away. Hatcheries, as a solution for lost spawning grounds, are not only expensive but unreliable when it comes to maintaining an important fish run.

The foregoing covers the major water uses except for water supply and pollution abatement. With the continued population growth and increased water requirement, the need for domestic and industrial water supply has become as serious in many parts of the country as the agricultural requirements. Similarly, the problem of pollution abatement has multiplied as the quantities of stream flows decrease and the amounts of industrial waste increase. Fortunately, the Columbia River Basin has few problem areas of this nature, either at the present time or in immediate prospect. Much attention has been called to this problem by the recent drought of the Southwest and careful attention will be given to the matter in the current comprehensive review report, with the assistance of the Public Health Service.

An additional element of the Columbia River problem, involving as much national policy as engineering, is the Canadian aspect. About 15 percent of the headwater drainage area of the Columbia River lies in British Columbia. From this area comes about 29 percent of the average annual runoff of the river at its mouth. Consequently, major water use developments in Canada could have a material bearing on developments downstream in the United States. In 1948, there was little active interest in Canada for early utilization of the water resources of the Columbia River. Since that time an active interest has developed in Canada for development of its resource potentials. Largely prompted by an inquiry of the International Joint Commission relative to the Libby project, but intensified by the comparison of economic growth in Canada with that of the United States resulting from large amounts of low cost power, interest in Canada in a comprehensive plan for maximum resources development has grown to possibly exceed even that in the United States' portion of the basin. At the present time the Canadian Federal Government, the Provincial Government and both public and private utility

interests of British Columbia are exploring, estimating and actively studying various project potentials. United States groups are also interested if there can be worked out a procedure whereby they could receive some of the fruits of their Canadian investment within the United States.

Canadian planning is of serious concern to the United States because of the downstream effects on United States projects. One of the plans under active study involves the diversion of the Kootenay River at Canal Flats, directing all the Kootenay River runoff above that point, a runoff of about 2,000,000 acre-feet annually, down the Columbia River in Canada rather than down the Kootenai River through Libby and the other potential head developments within the United States. From the Canadian point of view that would be a far better plan, but from the United States point of view, many millions of kilowatt hours would be lost. The question is, which is best from the over-all point of view and then how should the loser be compensated?

Another plan, involving diversion in Canada from the Columbia River into the Fraser River Basin, would mean a complete annual loss of some 15,000,000 acre-feet of water to the United States through all its main stem plants—Grand Coulee and downstream. Although this proposal appears possible from an engineering standpoint, and possibly attractive from the economic viewpoint of Canada, it may not prove entirely practicable.

Of course, the major International problem will be the adjustment of costs and benefits. Any regulation in Canada will benefit the United States, provided the water isn't lost entirely. Coordinated operation of the projects of the two countries would result in a greater benefit. To be equitable about it, Canada and the United States should share these downstream benefits. The question is, how much to each? Naturally the interests on each side of the boundary want as much as they can get. Engineering will play its part in estimating the values involved, but the final decision probably will be made by others after many factors have been considered. Until this is done, the final economic picture for the United States projects remains nebulous. Yet development must go forward. The power and the flood control are needed, and needed now.

And so planning must proceed. The Columbia River must be controlled and if the maximum return is to be realized at a minimum cost, both in dollars and other resource values, all the engineering science available must be employed as well as good judgment. Realizing the magnitude of the task and the great responsibility it imposes, the Corps of Engineers is calling on the best talent available throughout the region and the country in preparing its report. The Corps' procedures have been likened to a "fish bowl" operation, since everyone is invited to look on and express opinions. All interested Federal and State agencies are participating. The Bureau of Reclamation is actually taking over a large share of the detailed project studies. Private power and public power people from both the administrative and the operational level are represented on the Corps' advisory committees. So are navigation and other interests. There is a board of engineering consultants from other parts of the nation. This is a departure from previous custom and while many complications and problems are added by reason of the extra cooks employed in the making of the broth, engineers who are familiar with this type of work will recognize that in this increasingly complicated world many things must be considered other than straight engineering on a technical level.

That is the broad picture of the many things involved in controlling the Columbia. It is obvious that comprehensive planning is needed and it is obvious, in spite of the fact that changes are almost inevitable, long-range planning must continue if the Columbia River is to be controlled effectively and to the maximum practicable advantage.

Paper 1515 January, 1958 WW 1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

HYDRAULIC PROBLEM SOLUTION ON ELECTRONIC COMPUTERS[a]

Edward A. Lawler and Frank V. Druml
(Proc. Paper 1515)

SUMMARY

The authors present applications on three types of electronic computers demonstrating the wide range of problems readily solved by these modern tools. Tedious, cumbersome, conventional methods of solution are replaced by quicker and frequently more accurate methods, thus making more efficient use of the engineer for truly engineering problems.

INTRODUCTION

Economical use of electronic computers becomes justified only in those instances where the problem solved falls into one or more of the categories (a) complex problems not readily solvable by conventional methods, (b) repetitive problems, not necessarily complex, (c) problems neither complex nor repetitive, but those where even a small saving in time or personnel may be important. Group (a) may be problems such as the solution of simultaneous equations, (b) unit hydrograph computations and (c) flood routing problems where predictions are used to determine the schedule of flood control projects.

The Ohio River Division of the Corps of Engineers have used three computers for the solution of hydraulic problems. Typical applications on each computer will be discussed. They represent in each case only a few of the problems readily solved by electronic computers.

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1515 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

a. Paper presented at a meeting of the American Society of Civil Engineers, Jackson, Miss., February, 1957.

Authors — Section on: Univac I by F. U. Druml, Louisville District, Corps of Engineers, Louisville, Ky.
GEDA and Burroughs E 101, by E. A. Lawler, Ohio River Division, Corps of Engineers, Cincinnati, Ohio.

Applications

A. Univac I.

The Remington-Rand Univac I, a large high speed digital computer, while not the most modern, largest or speediest available, may be considered as representative of its class.

In cooperation with the Institute of Mathematics and Mechanics of New York University a program was established to formulate a method of solution for the partial differential equations (1) expressing the fundamental law of unsteady flow. The equations

$$H_x + \frac{v}{g} V_x + \frac{i}{g} Vt + \frac{V^2}{C^2 R} = O \qquad (1)$$

$$AV_x + VA_x + BH_t = O \qquad (2)$$

for solution by numerical methods may be written

$$V_t + V V_x + g H_x = -GV\backslash V\backslash \qquad (3)$$

$$(VA)_x + BH_t = Q \qquad (4)$$

where H denotes elevation of water surface, V-velocity, A-cross sectional area of the stream, B-surface width of the stream, G-a friction function, Q-local inflow per unit length of the river and g-acceleration of gravity. Subscripts t and x denotes the derivative of the variable with respect to time and distance along the stream respectively. The friction function $G = \frac{gn^2}{(1.486)^2 R \frac{4}{3}}$, where n is Mannings n and R is hydraulic radius.

Replacing the derivatives by difference quotients transforms the equations into a system of difference equations. The integration of equations (3) and (4) can be performed if the coefficients A, B, G and Q (here including tributary inflow) are known and boundary conditions are established at end points.

Three problems were presented for solution - (a) movement of a flood wave in a river channel, (b) movement of a flood wave at the junction of two tributaries, and (c) movement of a flood wave in a long narrow reservoir. In general the same system of difference equations and method of solution was employed in all three problems.

The river channel chosen for the first problem was the 375 mile reach of the Ohio River from Wheeling, W. Va., to Cincinnati, Ohio. Co-efficients A, B and G were computed emperically from available records and boundary conditions prescribed at Wheeling were observed stage and discharge data. Boundary conditions at Cincinnati were prescribed as a rating curve. Local runoff and tributary contribution as observed were assumed known. Reproduction of the 1945 flood hydrographs at key points on the Ohio appears on Plate I. For comparative purposes both observed and computed hydrographs are shown. Solution by the method of finite differences implies the determination of approximate solutions of the differential equations in a discrete net of points in an x, t plane. Use was made of a staggered net, wherein $2x = 10$ miles and $t = 9$ minutes. Solution time, excluding coding of basic data, editing

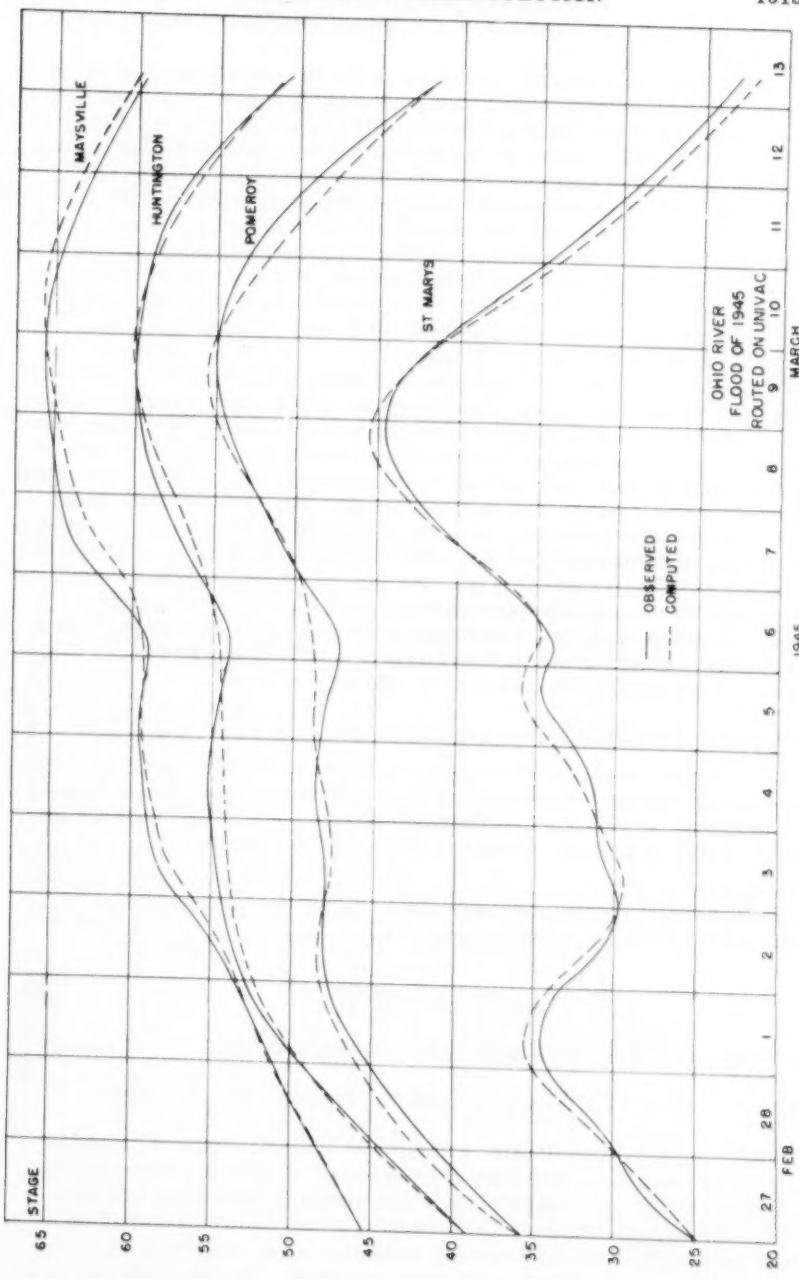

MAYSVILLE
HUNTINGTON
POMEROY
ST MARYS
OHIO RIVER
FLOOD OF 1945
ROUTED ON UNIVAC
OBSERVED
COMPUTED
STAGE
65
60
55
50
45
40
35
30
25
20
FEB
27
28
1
2
3
4
5
6
7
8
9
10
11
12
13
MARCH
1945

PLATE I

of results and input and output time, is 30 minutes for each 24 hour period, or 6-3/4 hours for 13-1/2 days.

The junction chosen was the confluence of the Ohio and Mississippi Rivers at Cairo, Illinois. Boundary conditions were established at Metropolis, Illinois, on the Ohio and Thebes, Illinois, on the Upper Mississippi, each approximately 40 miles above the confluence and at Hickman, Kentucky, on the Lower Mississippi approximately 40 miles below Cairo. During the computations certain equalities are requirements of the solution, such as; the sum of the flows from the Ohio Branch and the Mississippi Branch flowing into the junction must equal the flow out of the junction; at the junction elevation in all three branches must be identical. Plate II shows the routing of the January 1947 flood, observed and computed hydrographs being plotted. Computing time, excluding coding of basic data, time of input, output and editing, is about 3-1/2 hours for 20 days.

The long narrow reservoir chosen in the example was Kentucky Reservoir on the Lower Tennessee River. The reservoir is 184 miles in length, extending from Kentucky Dam to Pickwick Landing Dam. Boundary conditions were established at both ends of the reservoir in terms of discharge since both inflow at head of the reservoir and outflow at the lower end are fully controlled. Local and tributary inflow were assumed known. The 1948 and 1950 floods were routed thru the reservoir. Results of the 1950 flood routing are shown on Plate III, with observed data plotted for comparative purposes. The 1948 flood computations extended only until the date of February 1948, at which time the rate of change of elevation exceeded the limits of the method adopted. A modification in the method will eliminate this difficulty. Time of solution, again excluding coding time, time of input, output and editing, is approximately 12 minutes for each day, or 4 hours for 20 days.

B. Goodyear Electronic Differential Analyser

The Goodyear Electronic Differential Analyser or "GEDA" is a high quality analyser or "analog" employing high gain operational amplifiers. Procured originally in 1954 for flood routing problems, it has been used for the solution of other problems, several of which will be discussed.

1. Flood Routing

The Muskingum Equation [2] when expressed in differential form is readily solved by G.E.D.A. In the equation:

$$\frac{dO}{dt} = C_o (I - O) - C_1 \frac{dI}{dt} \qquad (5)$$

I and O are instantaneous values of inflow and outflow and

$$C_o = f_1 (O) = \frac{1}{K(1-x)} \text{ and } C_1 = f_2 (O) = \frac{x}{1-x} \quad \text{where}$$

K and X, determined emperically, express the characteristics of prismatic and wedge storage in a river channel as functions of outflow. In the solution of GEDA the known or predicted inflow hydrograph is introduced into the computed by a curve follower. Coefficients C_o and C_1 are developed by function generators from continuously computed values of outflow. Initial value of outflow is introduced as an initial condition. The computer continuously computes the rate of change of outflow and when integrated and

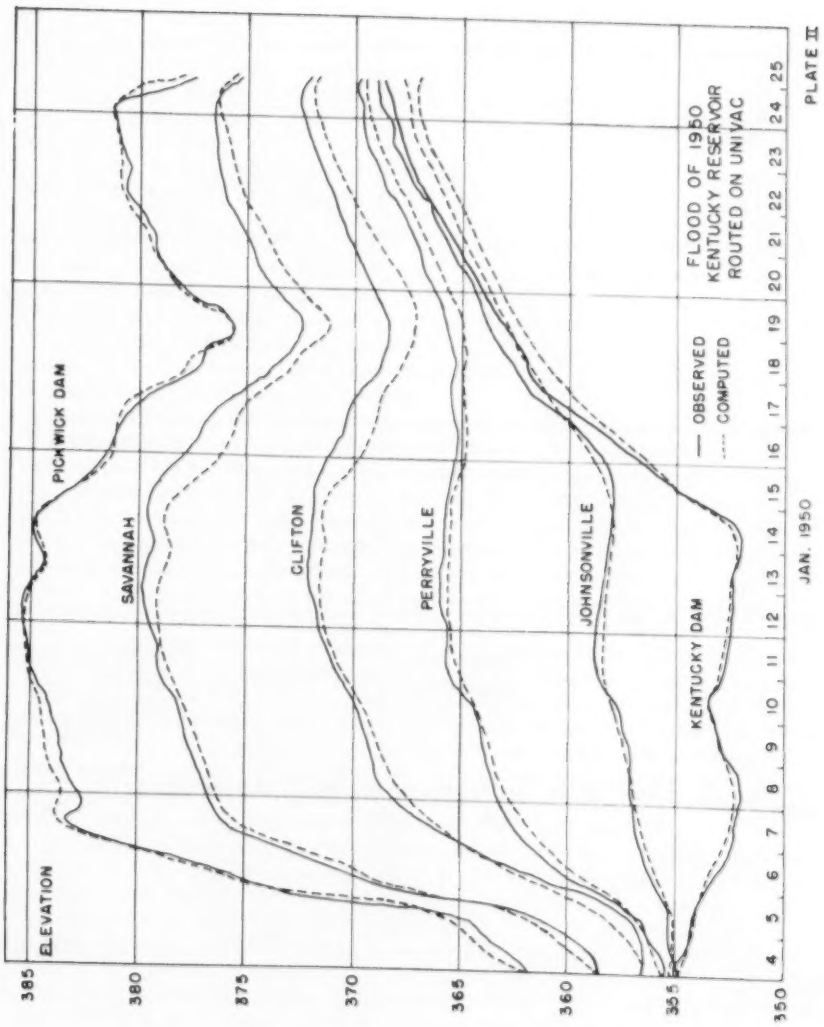

ELEVATION
385
380
375
370
365
360
355
350
PICKWICK DAM
SAVANNAH
CLIFTON
PERRYVILLE
JOHNSONVILLE
KENTUCKY DAM
4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25
JAN. 1950
— OBSERVED
---- COMPUTED
FLOOD OF 1950
KENTUCKY RESERVOIR
ROUTED ON UNIVAC

PLATE II

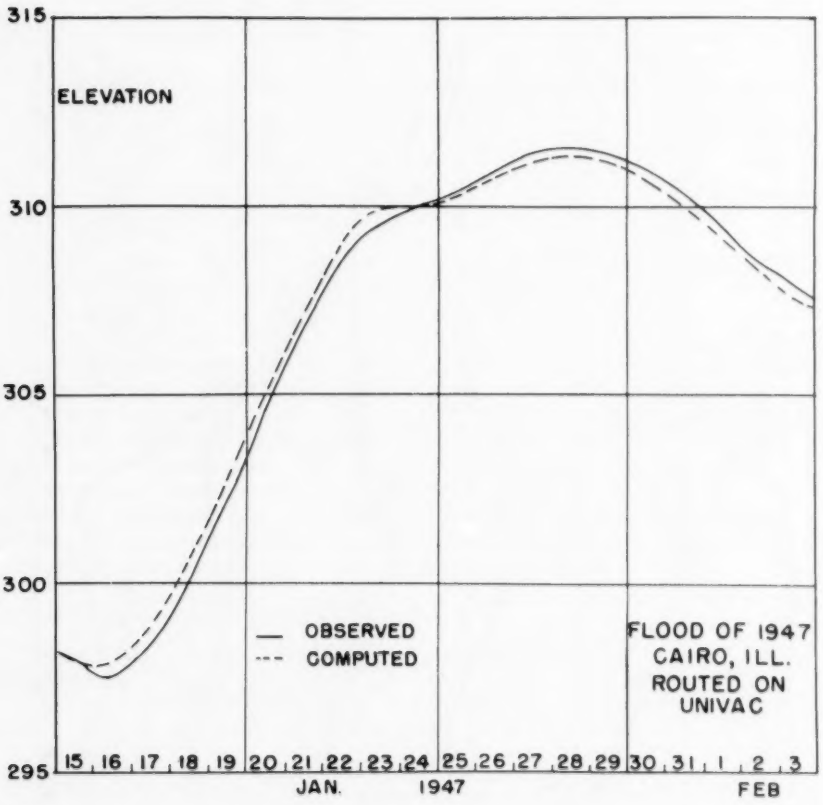

315
310
305
300
295
ELEVATION
OBSERVED
COMPUTED
FLOOD OF 1947
CAIRO, ILL.
ROUTED ON
UNIVAC
15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 1 2 3
JAN.
1947
FEB

PLATE III

recorded, results in a continuous hydrograph of outflow. The patch diagram, showing how the various computer components are inter-connected to satisfy the equation, is shown in Plate IV. Plate V is a recording of the routing of the 1937 flood thru the St. Marys-Pomeroy reach of the Ohio River. Total inflow into the reach, observed outflow, and arithmetical solution using finite time periods of one half day are also plotted for comparative purposes. Plate VI shows same plottings for the 1937 flood thru the Maysville-Cincinnati reach of the Ohio River. Plate VII is a plotting of the C_0 and C_1, functions of O.

Multiple reach routing can be accomplished by cascading. Since the outflow from the first reach becomes a portion of the inflow for the next reach it is combined with the local and tributary inflow in the second reach by the use of a second curve follower and a summing amplifier. Plate VIII shows the patch diagram for two reaches. Time of solution may be varied considerably, dependent upon the time scale chosen. The time scale of 5 seconds for 1 day has been found convenient. Time of solution for a 20 day flood, including setting of function generators, mounting of hydrographs, etc., is approximately 10 minutes. Computer running time is 100 seconds. Solution by conventional methods is about 2 hours.

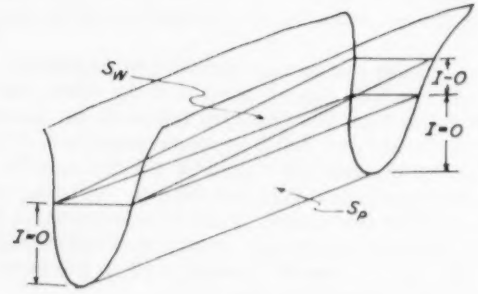
Simplification of the equation is frequently feasible, such simplification, when computations are made on desk calculator or by slide rule, materially reducing the time consumed. When employing G.E.D.A. time saving is also experienced, but to an even greater degree. Plate IX shows the patch diagram for the solution of the equation when the coefficients C_0 and C_1 are assumed as constants. This circuity permits the routing of 3 consecutive reaches concurrently. The time of solution, including mounting of charts, setting of coefficients etc., for a 20 day flood is about 5 minutes. Comparable time using conventional methods is about 3-1/2 to 4 hours.

In determining the benefits resulting from regulation of floods by reservoirs, a choice of several methods is available. One method is to route the reductions, or the differences, plus and minus, between the natural and modified inflow hydrographs, and applying the routed reductions to the natural outflow hydrograph. Another method is to route the natural hydrograph and the modified hydrograph. Either of these methods can be employed on G.E.D.A. with ease. Plate X shows the patch diagram for routing reservoir reductions thru 5 reaches, using 2 constant coefficients for each reach.

In certain studies the effects resulting from reservoir regulations is desired at a point several reaches below the initial inflow point. Plate XI shows a patch diagram for such a solution. Nine consecutive reaches are routed and the results may be recorded at any or all of the nine outflow points. This represents the circuit for routing the effects of the Pittsburgh District Reservoirs from Wheeling, W. Va. to Metropolis, Ill. near the mouth of the Ohio, a distance of almost 900 miles. Plate XII shows a typical recording. Reductions at Wheeling, W. Va. and all nine outflow points were recorded. Time of solution is about 10 minutes, computer running time for this flood of 40 days being 40 seconds. Time required by conventional methods is not less than 8 hours.

2. Surges in Sewers

At local protection projects, where gravity flow in sewers in interrupted during flood periods and pumping is required, the surges resulting from modification of pump operation or power failure must be evaluated.


S_W
I-O
I-O
I-O
S_p

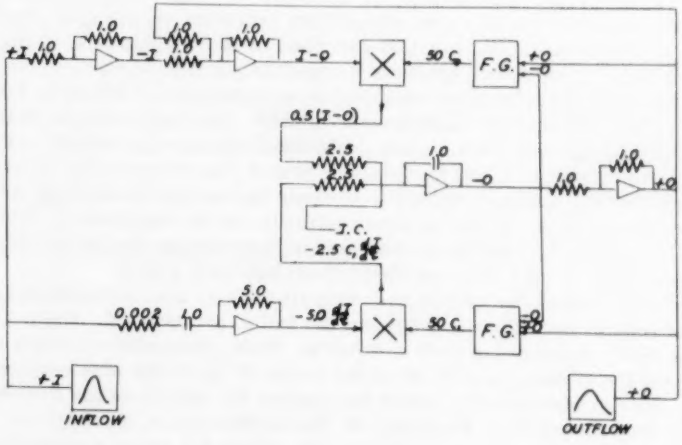

+I
1.0
-I
I-O
50 C
F.G.
+O
0.5(I-O)
2.5
5
-O
I.C.
-2.5 C
0.002
5.0
-50
INFLOW
OUTFLOW

PLATE IV

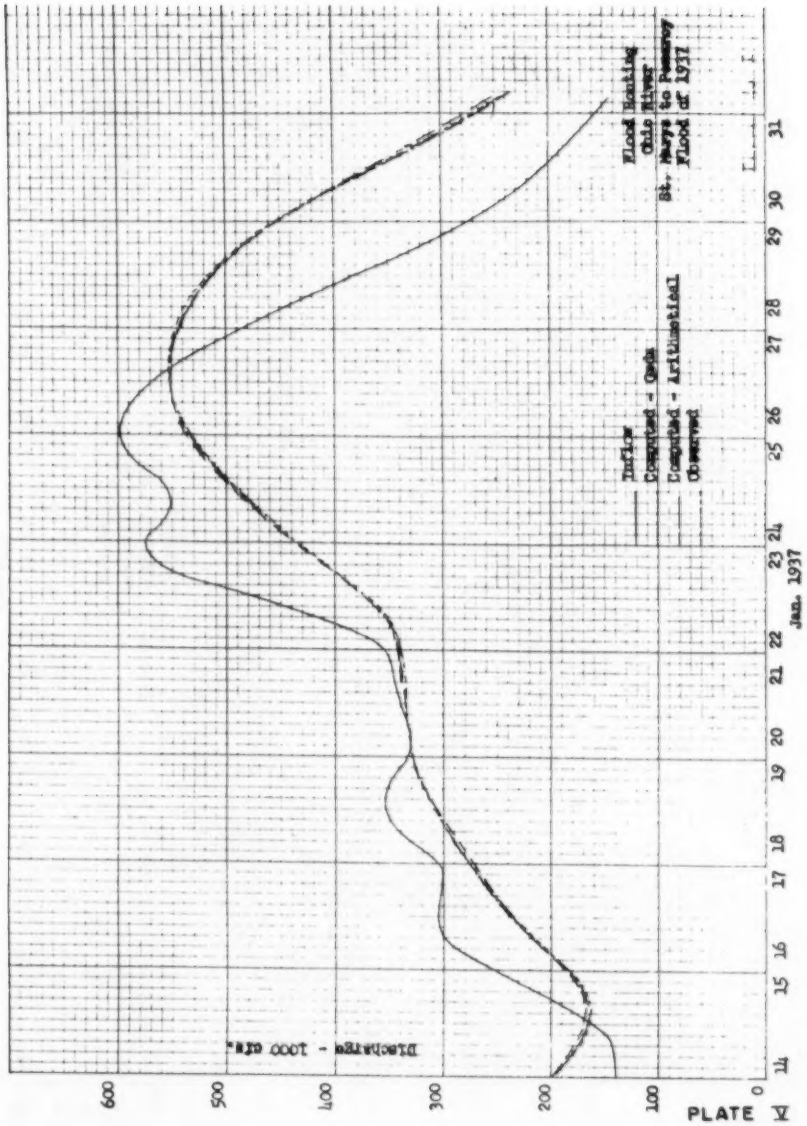

Flood Routing
Ohio River
St. Marys to Pomeroy
Flood of 1937
Inflow
Computed - Gwdn
Computed - Arithmetical
Observed
Discharge - 1000 cfs.
600
500
400
300
200
100
0
14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
Jan. 1937

PLATE IX

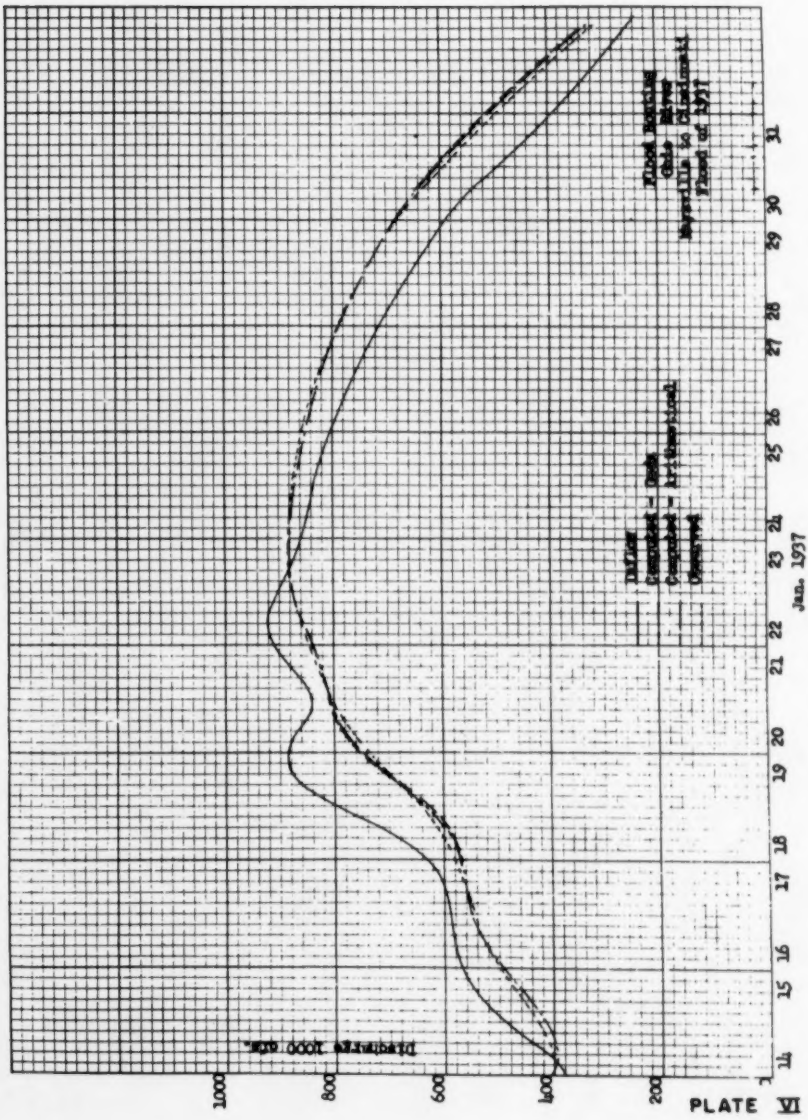

1000
800
600
400
200
1
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
Jan. 1937

PLATE XI

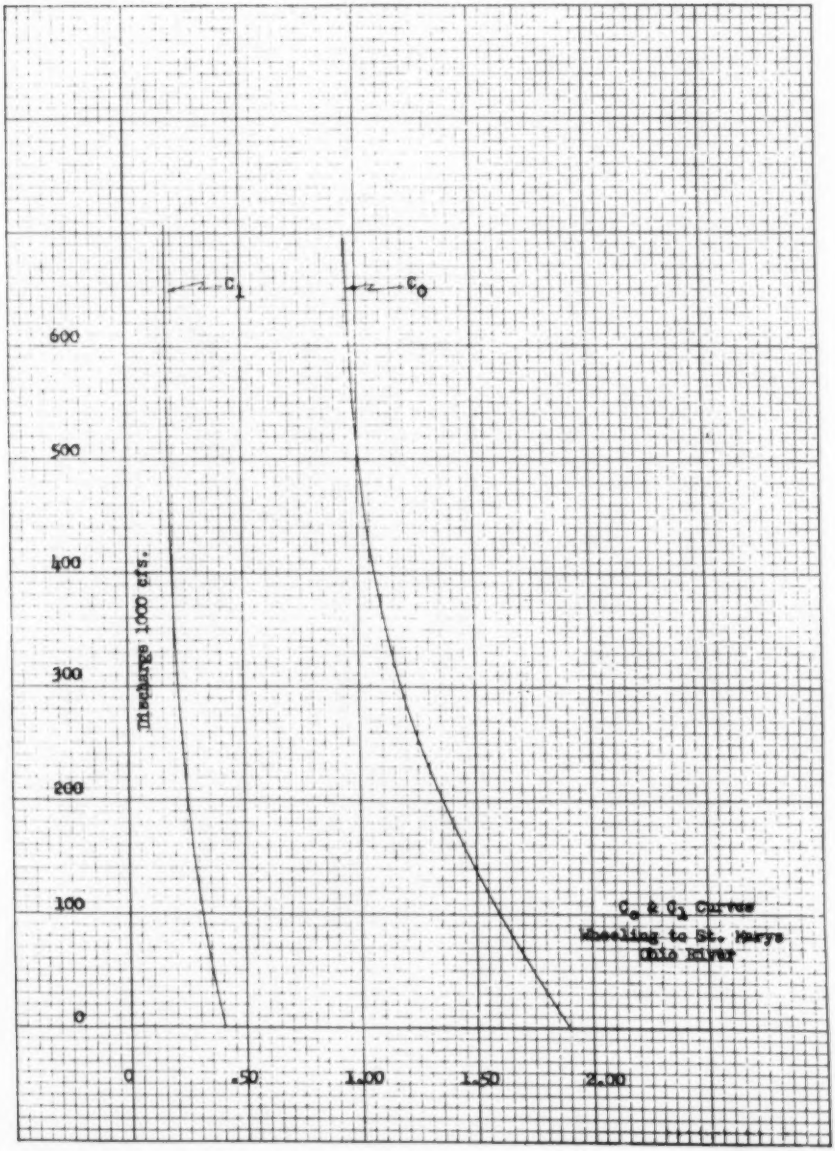

C_1
C_0
600
500
400
300
200
100
0
Discharge 1000 cfs.
0
.50
1.00
1.50
2.00
C_0 & C_1 Curves
Leading to St. Marys
Ohio River

PLATE VII

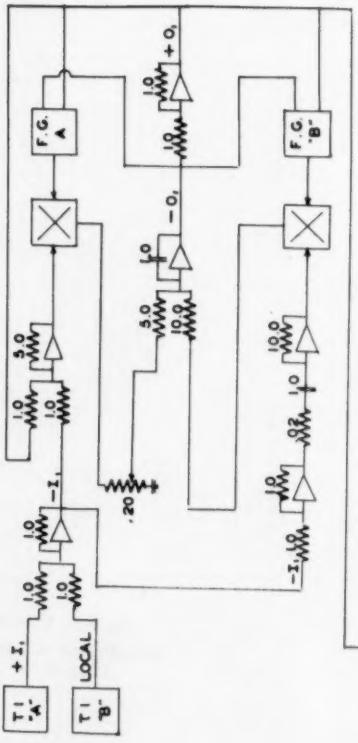


PLATE VIII

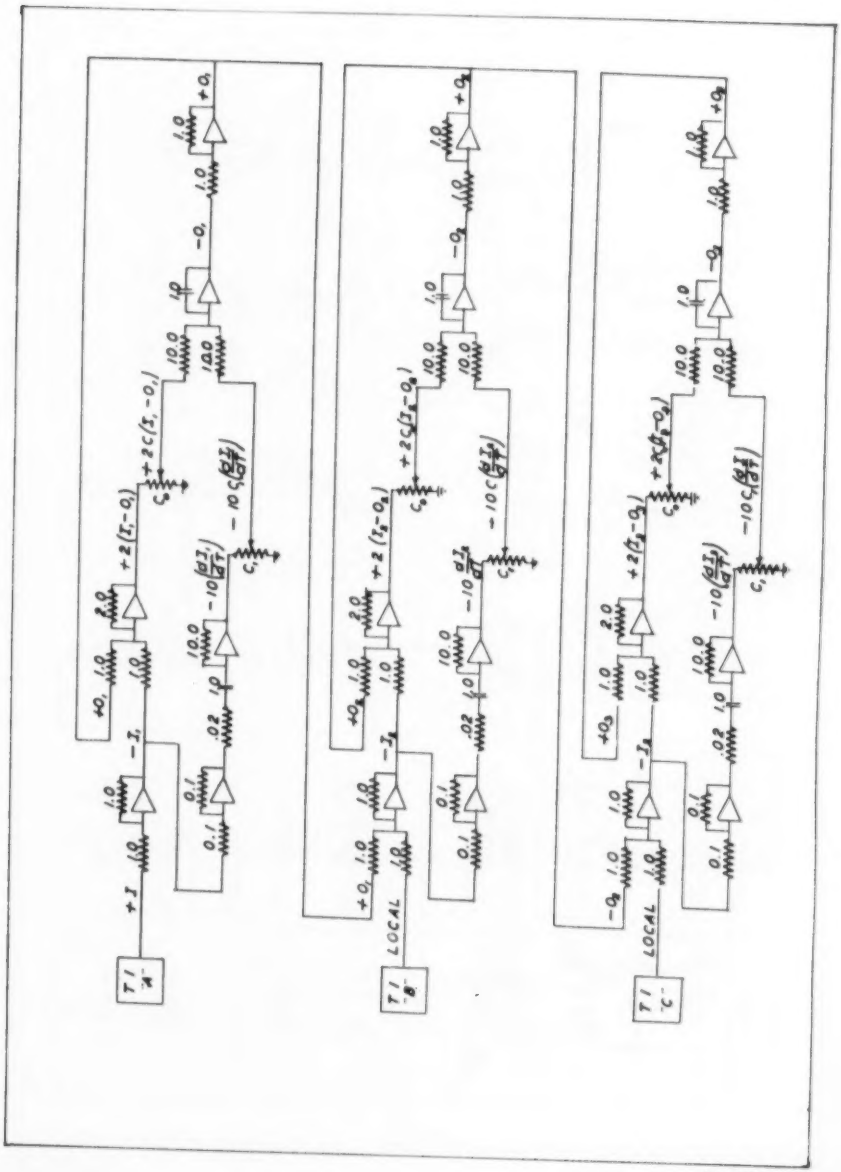


PLATE IX

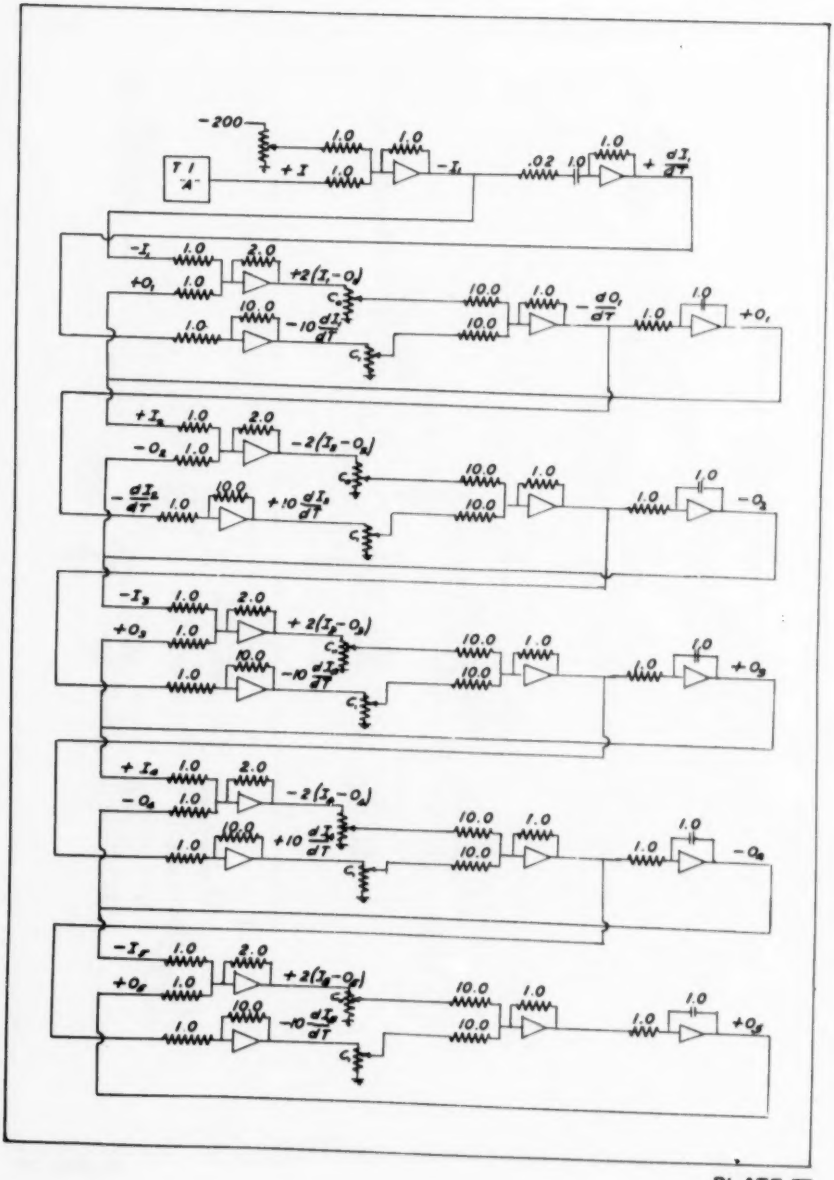

-200
1.0
1.0
T I "A"
+ I
1.0
-I₁
.02
1.0
1.0
+ dI₁/dT
-I₁
1.0
2.0
+2(I₁-O₁)
+O₁
1.0
C₁
10.0
1.0
- dO₁/dT
1.0
1.0
+O₁
10.0
-10 dI₁/dT
1.0
10.0
C₁
+I₂
1.0
2.0
-2(I₂-O₂)
-O₂
1.0
C₂
10.0
1.0
1.0
-O₂
- dI₂/dT
10.0
+10 dI₂/dT
1.0
10.0
C₂
-I₃
1.0
2.0
+2(I₃-O₃)
+O₃
1.0
C₃
10.0
1.0
1.0
+O₃
10.0
-10 dI₃/dT
1.0
10.0
C₃
+I₄
1.0
2.0
-2(I₄-O₄)
-O₄
1.0
C₄
10.0
1.0
1.0
-O₄
10.0
+10 dI₄/dT
1.0
10.0
C₄
-I₅
1.0
2.0
+2(I₅-O₅)
+O₅
1.0
C₅
10.0
1.0
1.0
+O₅
10.0
-10 dI₅/dT
1.0
10.0
C₅

PLATE I

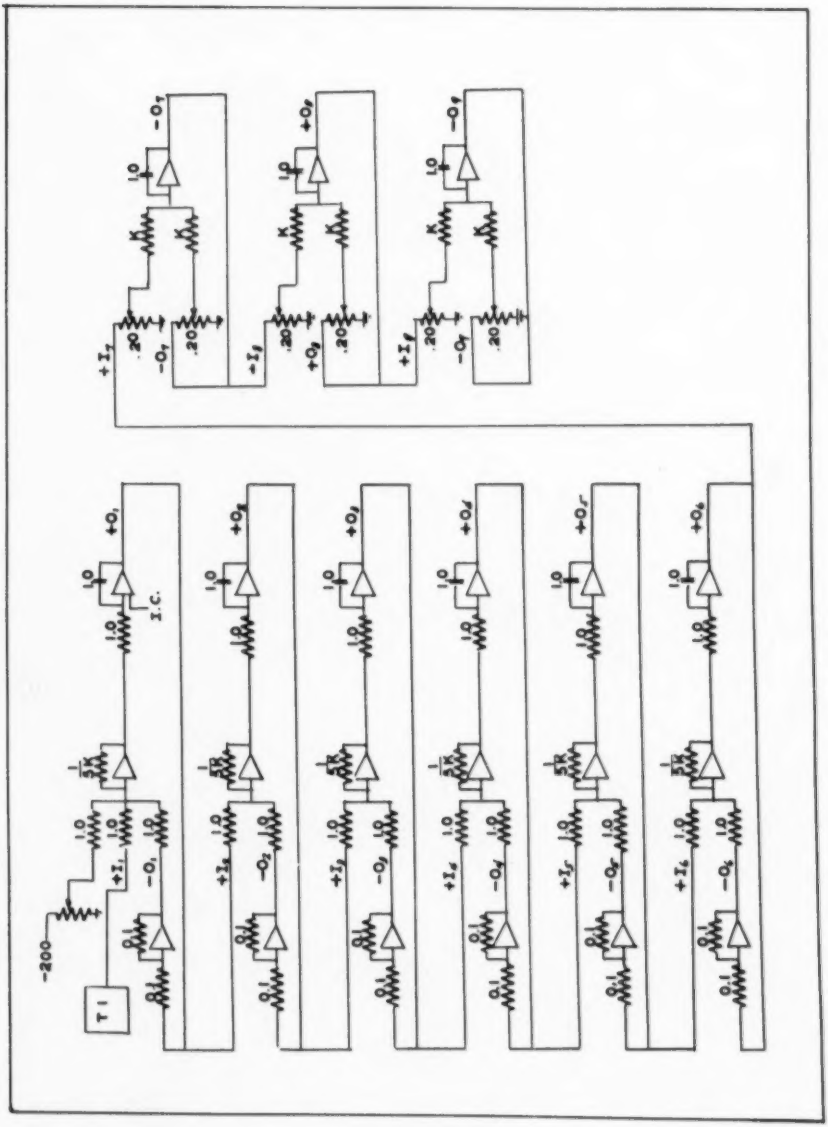
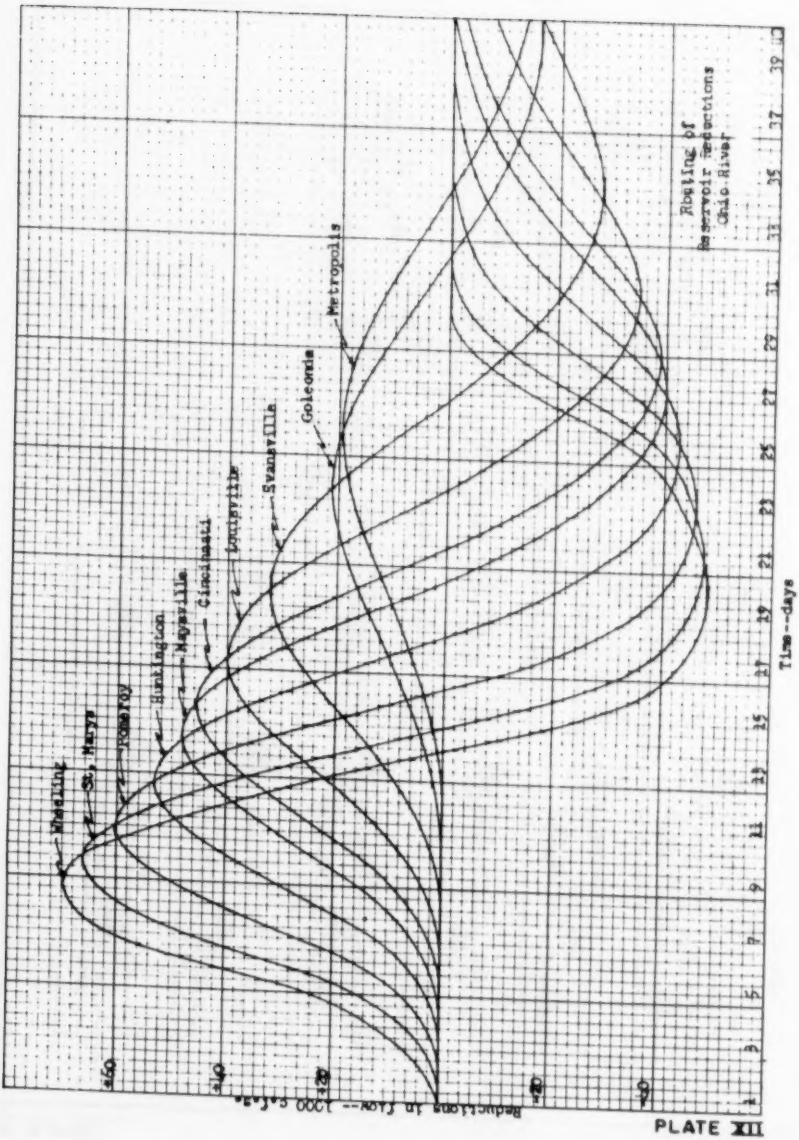


PLATE XI


Routing of
Reservoir Reductions
Ohio River
Wheeling
St. Marys
Pomeroy
Huntington
Maysville
Cincinnati
Louisville
Evansville
Golconda
Metropolis
Time--days
Reductions in flow -- 1000 c.f.s.
PLATE XII

Compatable surge range, sump and surge chambers must be determined, particularly with respect to economic considerations. A typical system is shown schematically in Plate XIII. Riser 1 is the pump sump and risers 2 to 5 surge chambers or storage areas in the system. Riser 6, shown as "Constant level reservoir" is the balance of the system, at such a distance as to have little or no effect on surges in the pump sump. I_2 to I_5 are lateral inflow points.

From the law of motion it can be shown that if an unbalanced head is applied to a mass of water of cross sectional area "a" and length "L" the resulting change in velocity will be gh/L and rate of change of discharge will be gha/L. From this relationship is developed the following:

$$\frac{dQ_1}{dt} = \frac{ga_1}{L_1}(Z_1 - Z_2 - C_1Q_1) \tag{6}$$

$$\frac{dZ_1}{dt} \quad \frac{Q_1 - Q_o + I_1}{A_1} \tag{7}$$

where Q is discharge

g is force of gravity

Z is elevation of water surface

I is lateral inflow

A is cross sectional area of riser

a is cross sectional area of sewer

L is length of sewer section

C is hydraulic loss factor, defined such as that CQ is total loss in sewer section.

Subscripts in equations refer to numbered sections. Pertinent data are given in Tables I and II. In the solution the following assumptions were made:

a. Head loss in sewer sections varies linearly with rate of flow.

b. Lateral inflow varies as square root of head.

c. Section shown as "constant level reservoir" is the balance of system at a distance greater than 24,415 feet from sump, and elevation assumed to be constant. Elevation in this section taken as reference datum.

In the patch diagram, Plate XIV, provision was made for readily changing the cross sectional area of the risers #1 and #2. Recordings were made of surges under varying conditions of pump operation and various sizes of pump sump and storage area. These are shown in Plates XV to XVIII. The time scale chosen was 1 second in solution equal to 10 seconds in problem. Time of each recording of 800 seconds was therefore 80 seconds. It is conservatively estimated that the time saving, as compared to conventional methods is in the order of 400 to 1.

3. Fluctuations in Pool Elevations

Improperly coordinated power releases may cause severe fluctuations at harbor sites in navigation pools on streams where power generation is one of

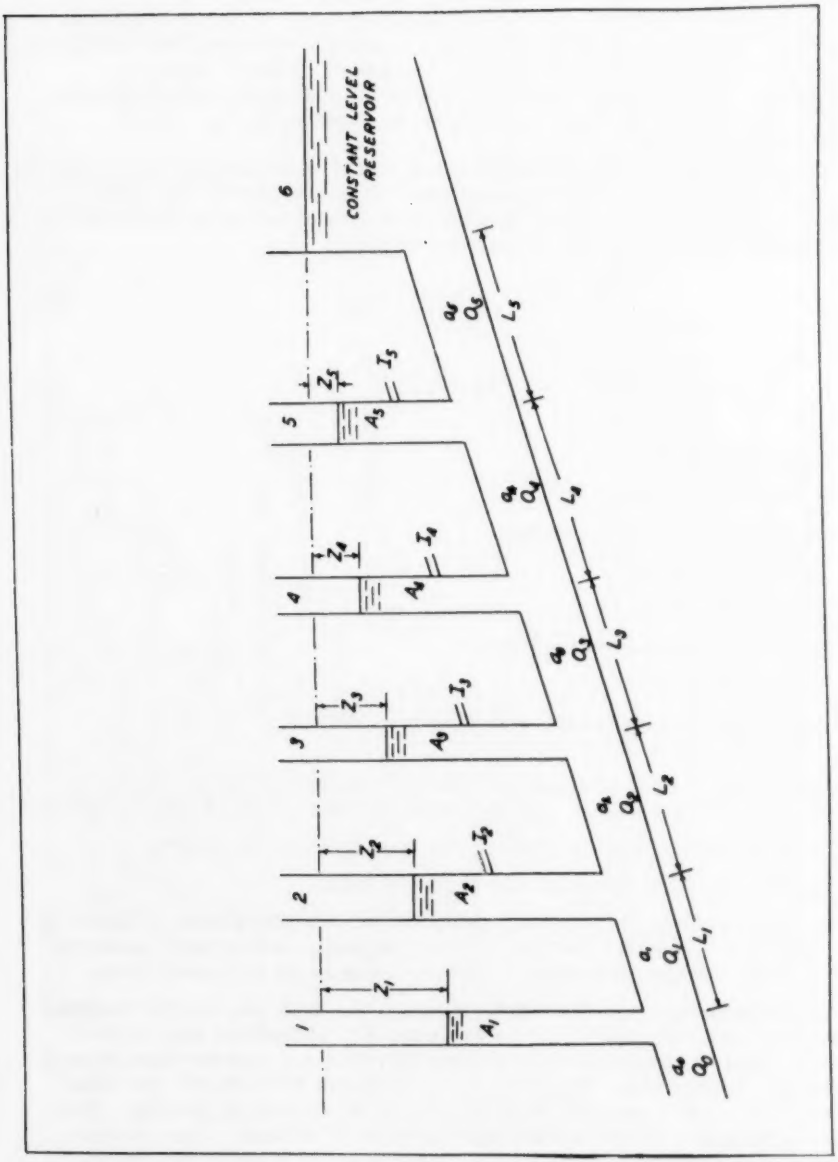

CONSTANT LEVEL RESERVOIR
6
5
4
3
2
1
Z_5
I_5
A_5
Z_4
I_4
A_4
Z_3
I_3
A_3
Z_2
I_2
A_2
Z_1
A_1
Q_5
Q_4
Q_3
Q_2
Q_1
Q_0
L_5
L_4
L_3
L_2
L_1

PLATE XIII

Table I
Pertinent Data - Horizontal Sections

Section Number	Area (a) Sq. ft.	Length (L) ft.	Discharge (Q)* cu. ft./sec.	Loss Factor (c) ft.
0	90.2	—	1800	—
1	90.2	1537	1800	0.00667
2	180.0	6143	1590	0.00730
3	150.0	5028	1170	0.00368
4	140.0	5072	940	0.00372
5	115.0	6635	530	0.01000

Table II
Pertinent Data - Vertical Sections

Riser Number	Area (A) sq. ft.	Water Surface Elev.* ft. above sea level	Head (Z)* ft.	Lateral Inflow (I)* cu. ft./sec.
1 (Pump Sump)	2100	416.0	-36.7	—
2 (Storage Area)	90000	428.0	-24.7	210
3	2350	439.6	-13.1	420
4	7350	443.9	- 8.8	230
5	350	447.4	- 5.3	410
6	—	452.7	0	—

*Steady flow conditions, prior to pump or power failure.

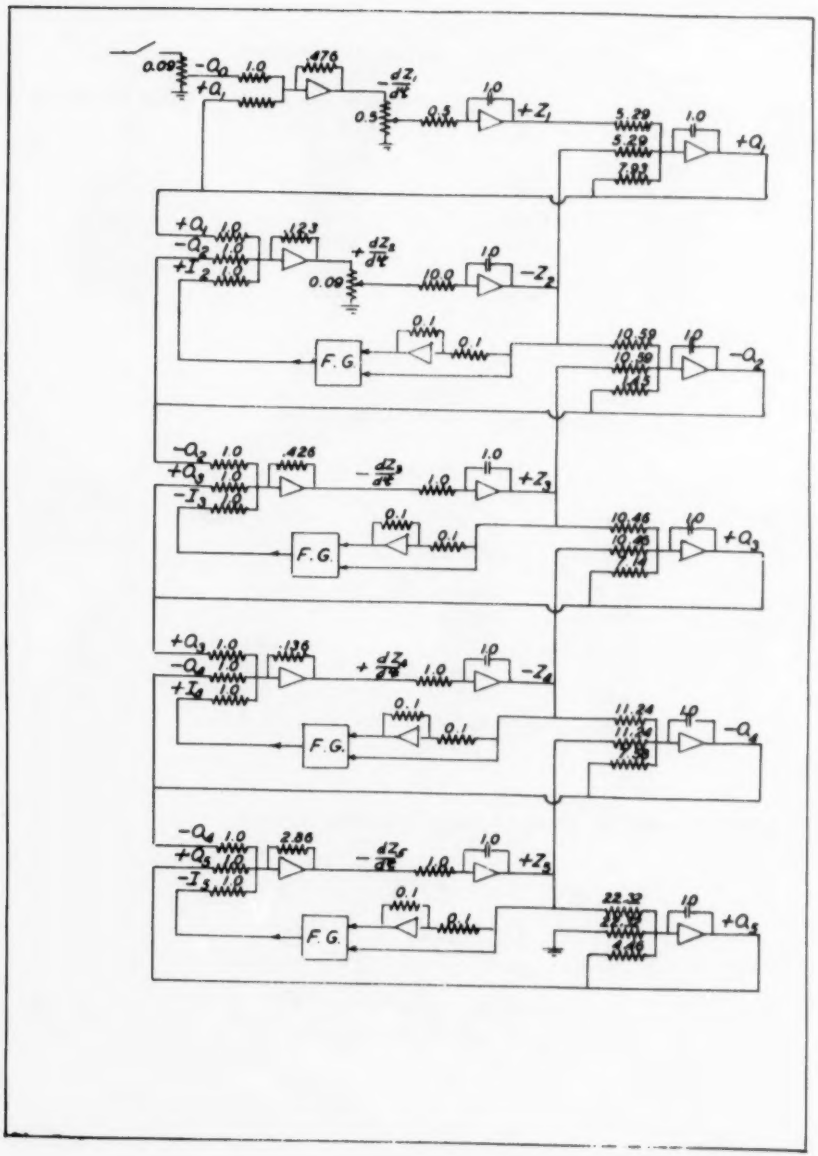

0.09
-Q0
+Q1
1.0
476
-dZ1/dt
0.5
0.3
+Z1
5.29
5.29
7.93
+Q1
+Q2
-Q2
+I2
123
+dZ2/dt
0.09
10.0
-Z2
F.G.
0.1
10.39
10.39
-Q2
-Q3
+Q3
+I3
426
-dZ3/dt
+Z3
10.46
10.46
+Q3
+Q4
-Q4
+I4
136
+dZ4/dt
-Z4
11.24
11.24
-Q4
-Q5
+Q5
+I5
2.86
-dZ5/dt
+Z5
22.32
22.32
+Q5

PLATE XIV

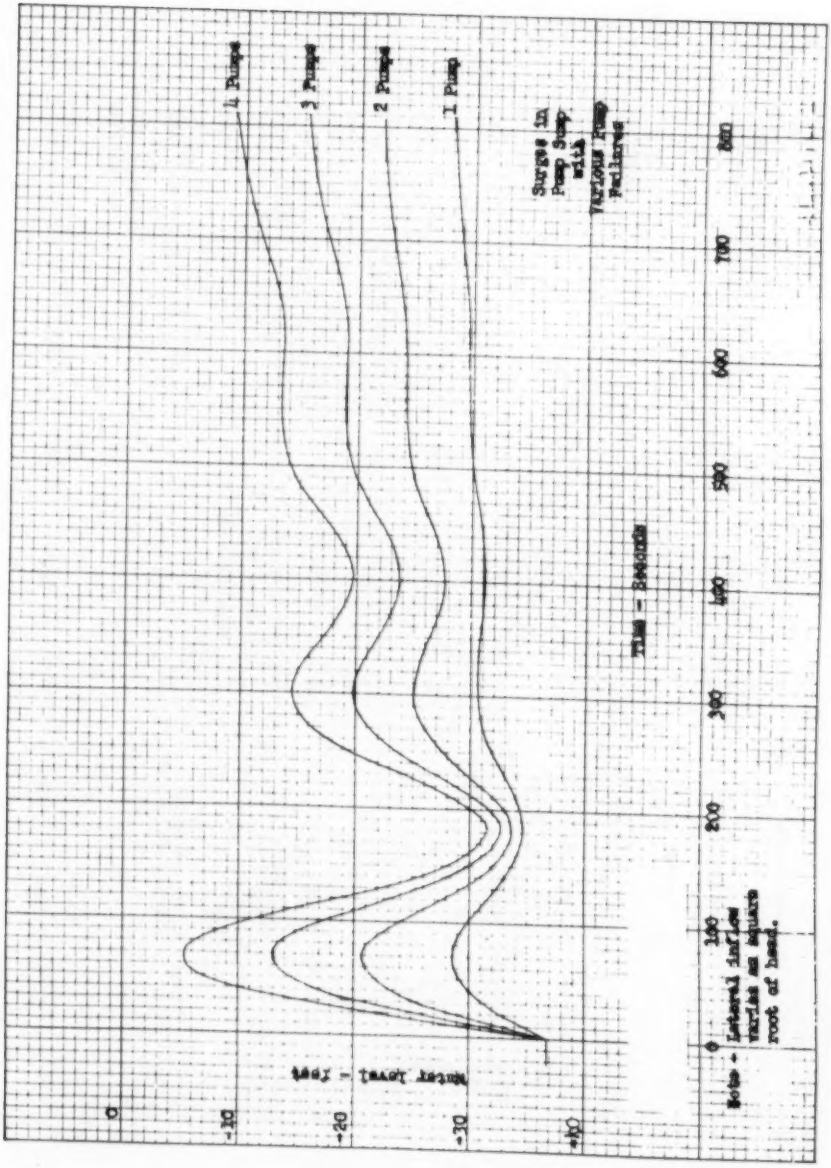

4 Pumps
3 Pumps
2 Pumps
1 Pump
Surge in Pump Sump with Various Pump Failures
Time - Seconds
Water Level - Feet
0
-10
-20
-30
100
200
300
400
500
600
700
800
Note - Lateral inflow varies as square root of head.

PLATE XV

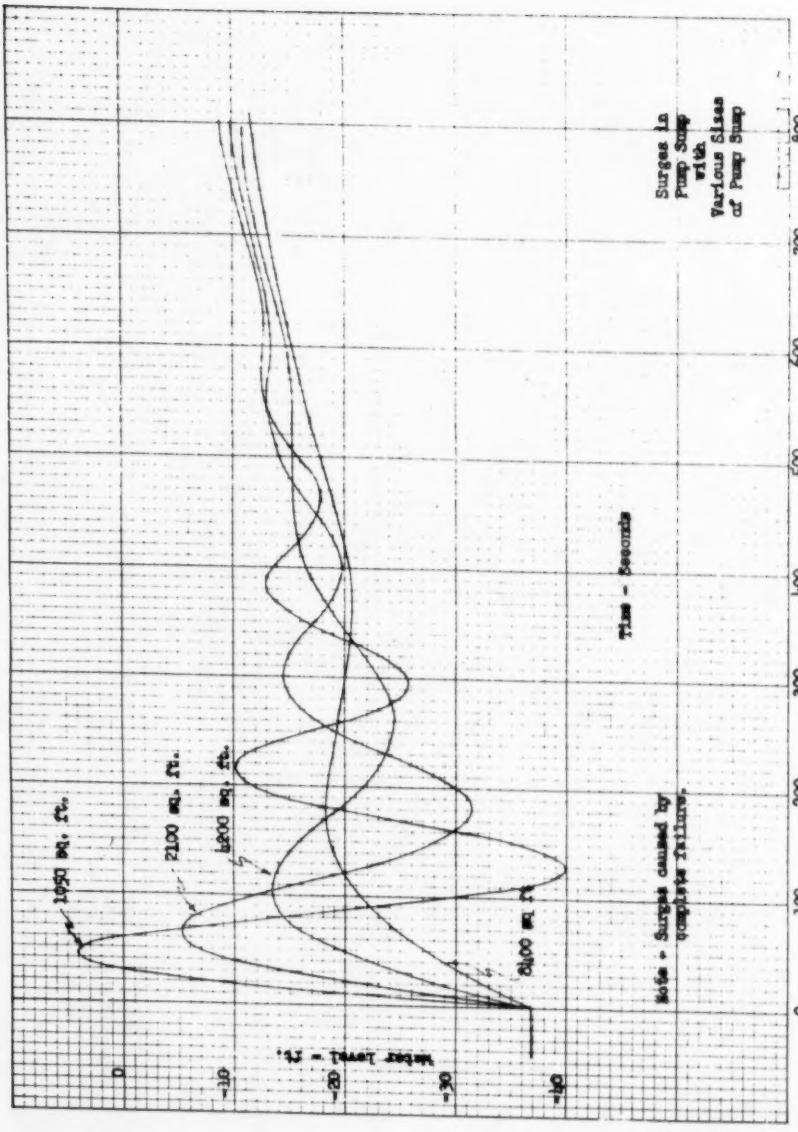

Surges in Pump Sump with Various Sizes of Pump Sump
Time - Seconds
Note - Surges caused by complete stoppage of pumps.
Water Level - Ft.
0
-10
-20
-30
-40
100
200
300
400
500
600
700
800
1950 Sq. Ft.
2100 Sq. Ft.
2400 Sq. Ft.
3000 Sq. Ft.

PLATE XVI

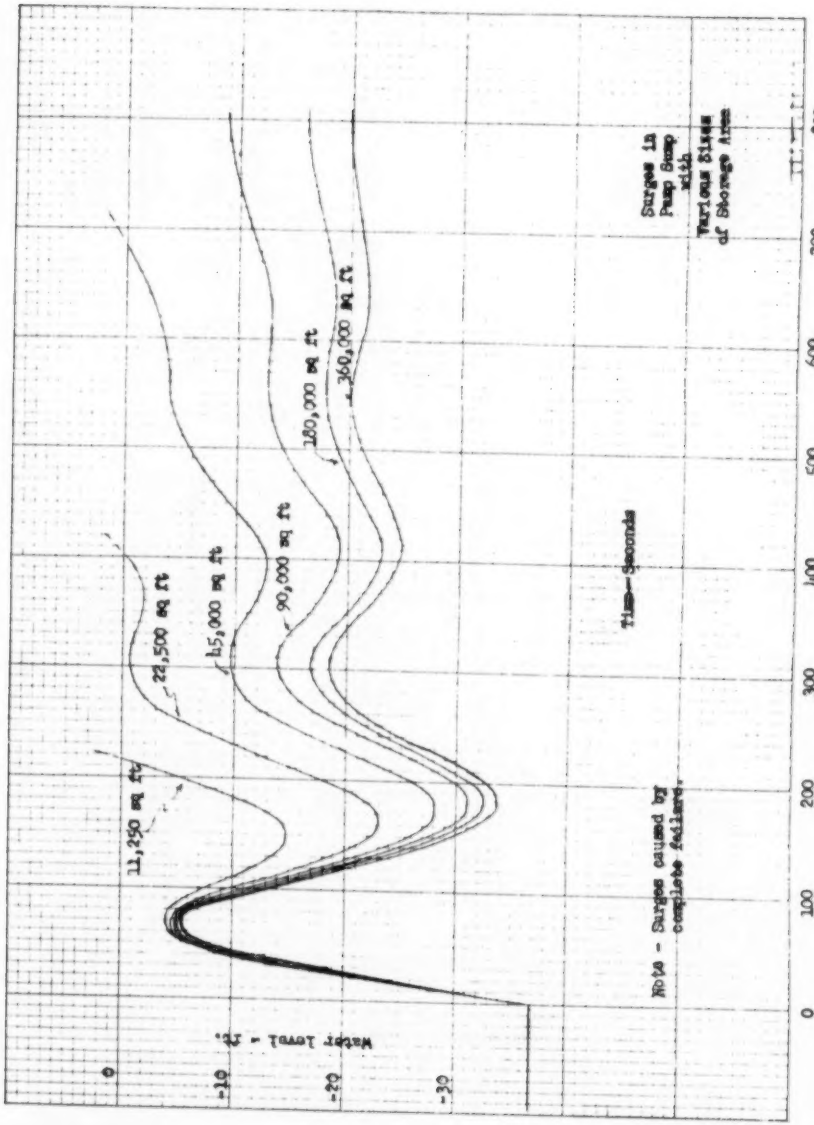

Surges in Pump Sump with Various Sizes of Storage Area
Time—Seconds
Water Level - Ft.
0
-10
-20
-30
0
100
200
300
400
500
600
700
800
11,250 sq ft
22,500 sq ft
45,000 sq ft
90,000 sq ft
180,000 sq ft
360,000 sq ft
Note - Surges caused by complete failure.

PLATE XVII

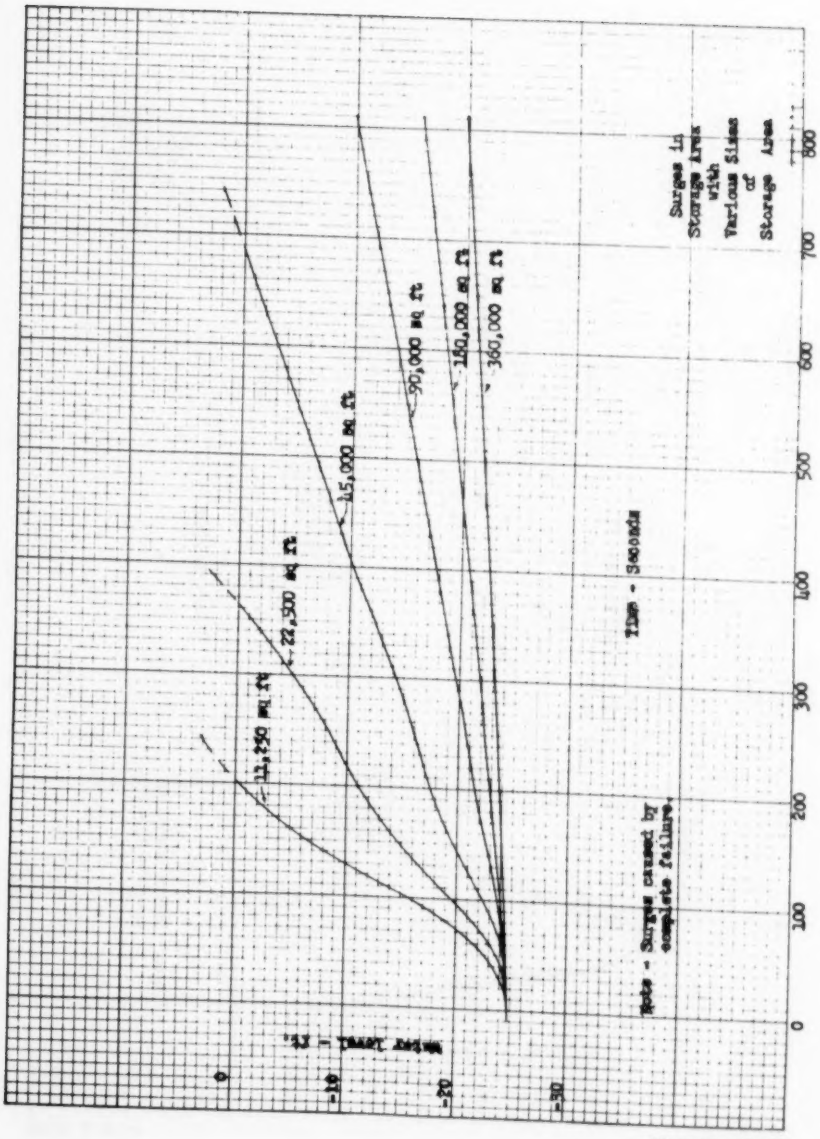

Surges in Storage Area with Various Sizes of Storage Area
Time - Seconds
Note - Surges caused by complete failure.
12,250 sq ft
22,500 sq ft
45,000 sq ft
90,000 sq ft
180,000 sq ft
360,000 sq ft
0
100
200
300
400
500
600
700
800
-10
-20
-30

PLATE XVIII

the purposes of the dams. Plate XIX shows schematically the location of a harbor in a navigation pool. Power releases from the upstream and downstream dam are made for generation of power during peak load periods. Maximum releases (power) at the upstream and downstream dams are 30,000 cfs and 20,000 respectively. The equations describing the movement of waves thru the reservoir are:

$$\frac{dS_{1-2}}{dt} = Q_1 - Q_2$$

$$\frac{dH_2}{dt} = \frac{Q_1 - Q_3}{A_{1-3}}$$

$$\frac{dQ_2}{dt} = \frac{ga_2}{L_2} (H_1 - H_3 h_2)$$

$$\frac{H_1 + H_2}{2} = BS_{1-2} + O$$

where S is storage in reach

Q is rate of flow

g is force of gravity

a is mean vertical cross sectional area of stream

H is elevation

L is length of reach

h is friction head loss $= L \left(\frac{1}{1.486\ r^{2/3}}\right)^2 Q^2$

A is mean horizontal area of stream

B & C are storage elevation coefficients

n is Mannings n

r is hydraulic radius

Subscripts relate to the sections under consideration. Boundary conditions of discharge are known or assumed. Patch diagram given in Plate XX.

Trial runs were made with various schedules of release covering the range normally anticipated. The fluctuations at the harbor site resulting from each schedule was recorded. Plate XXI shows the recordings, the number associated with each being the schedule listed in the table. It is estimated that the time saving ratio between GEDA solution and conventional methods is about 100 to 1. Limitations in components required certain simplifying assumptions and limited the number of sections to five. Additional components would produce solutions upon which a greater degree of confidence could be placed. However, it is believed that the results obtained are quite indicative.

Among the additional problems which have been solved on the Ohio River Division G.E.D.A. are the following:

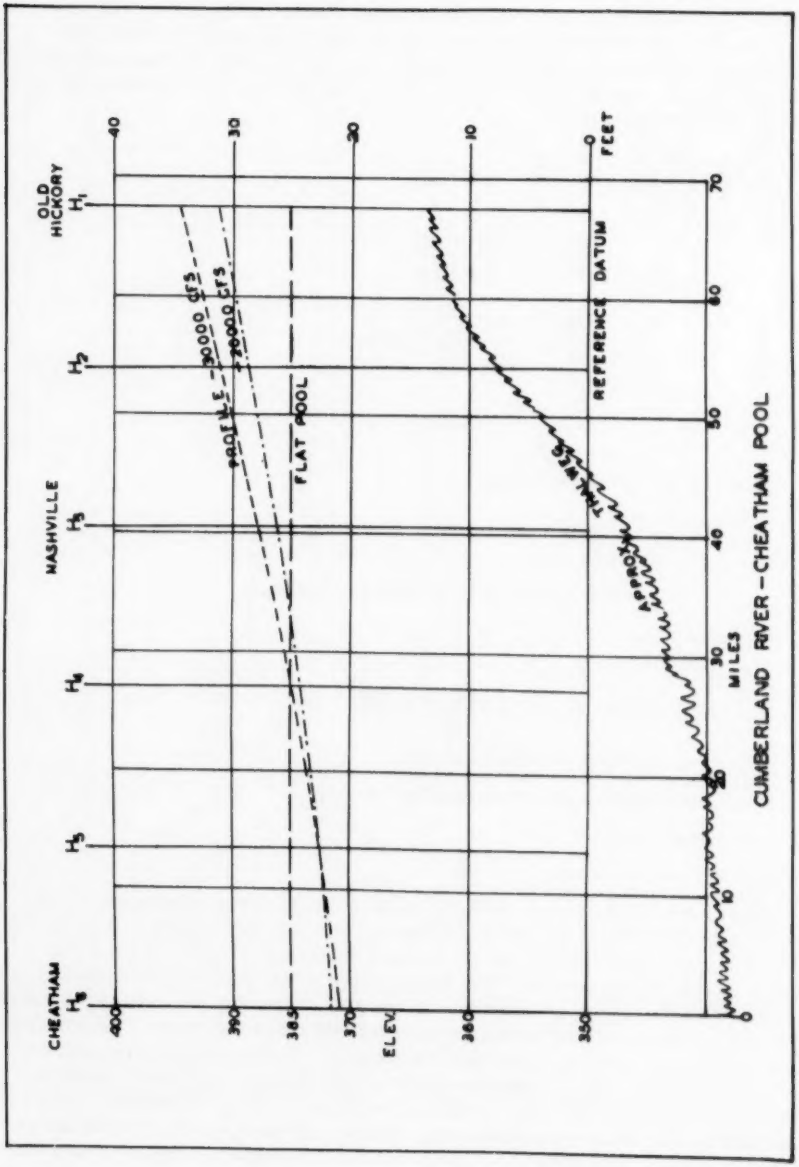

CUMBERLAND RIVER – CHEATHAM POOL
OLD HICKORY
NASHVILLE
CHEATHAM
H1
H2
H3
H4
H5
H6
PROFILE – 30000 CFS
PROFILE – 20000 CFS
FLAT POOL
REFERENCE DATUM
APPROX. THALWEG
ELEV.
FEET
MILES
400
390
380
370
360
350
40
30
20
10
0
70
60
50
40
30
20
10
0

PLATE XIX

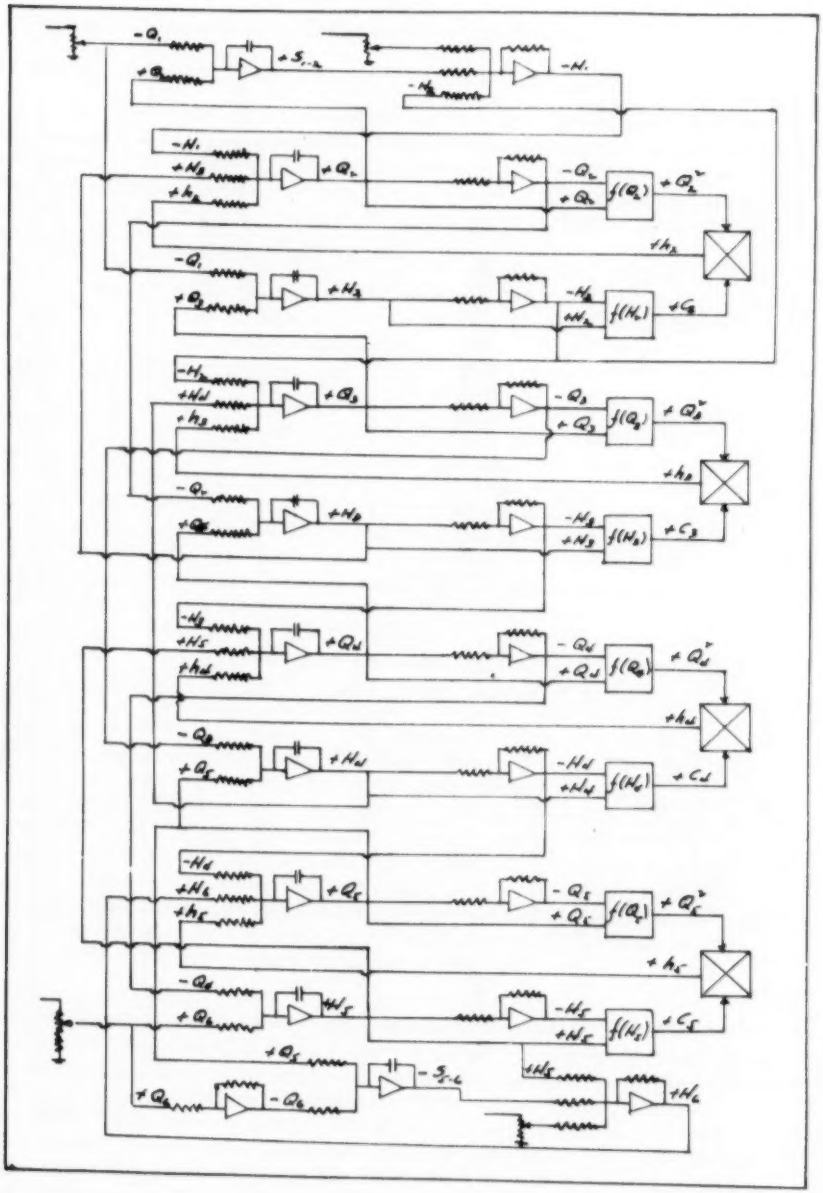


PLATE XX

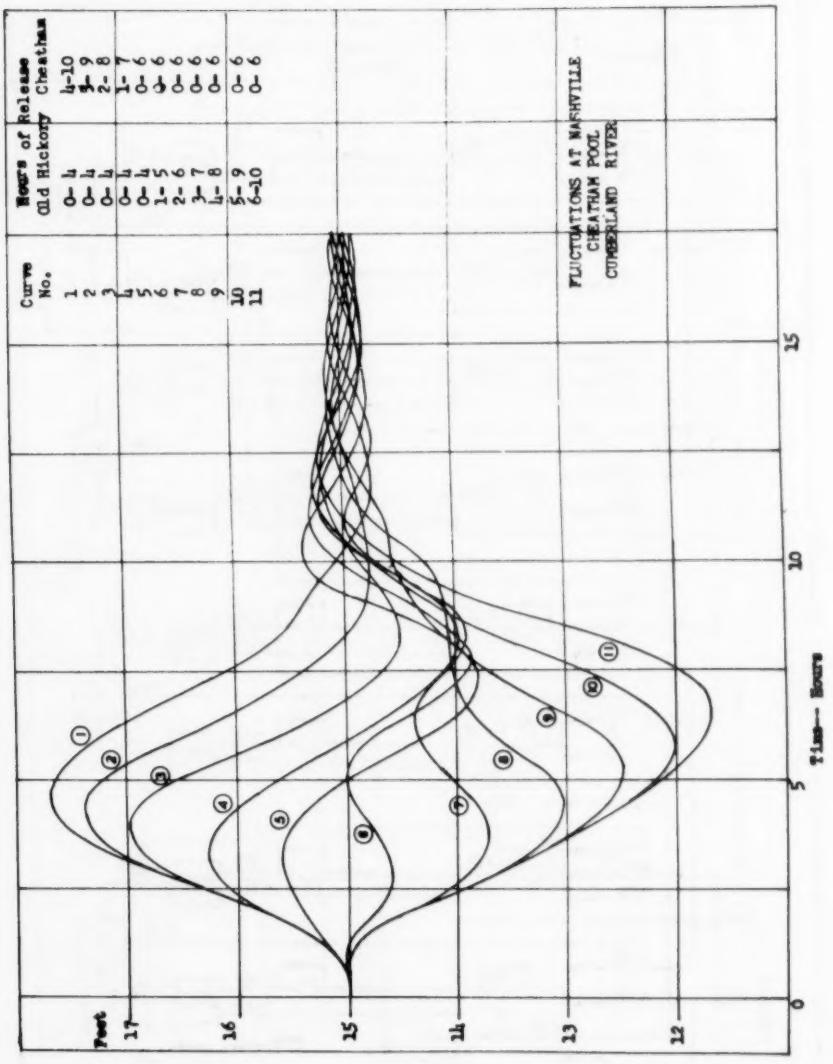

Curve No.
Hours of Release Old Hickory Cheatham
1 0-4 4-10
2 0-4 3-9
3 0-4 2-8
4 0-4 1-7
5 0-4 0-6
6 1-5 0-6
7 2-6 0-6
8 3-7 0-6
9 4-8 0-6
10 5-9 0-6
11 6-10 0-6
FLUCTUATIONS AT MARTVILLE
CHEATHAM POOL
CUMBERLAND RIVER
Feet
17
16
15
14
13
12
0
5
10
15
Time-- Hours

PLATE XII

Analysis of Siphon Air Vents

Analysis of Fluctuation of Hydro Power Surge Chambers

C. Burroughs E-101

The Burroughs E-101 is a moderate speed, small size digital computer. Data is entered by means of an eleven digit keyboard, similar to a desk calculator. The computer is externally programmed by means of 8 pinboards each having 16 instructions. One hundred words of memory are provided on a magnetic drum, each word consisting of 12 decimal digits plus sign. Printed output is on 16 inch 10 column paper, in accordance with any desired format. Tape input and output is obtainable as an optional accessory as is also an expanded memory of 120 additional words. Procured primarily for unit hydrograph computations this computer is being used for solution of a widely expanding range of problems.

1. Unit Hydrograph Computations

In connection with determining the flows from historical storm patterns, in place or transposed, computations must begin with rainfall data. The unit hydrograph (3, 4) describes the time distribution of flow from a given drainage area resulting from an effective rainfall of one inch for some unit of time duration. Expressed in terms of flow, the ordinates when applied to successive storm rainfalls produce the flows resulting from those rainfall amounts. The number of ordinates may vary from as few as 4 or 5 to as many as 30 or more, depending upon the drainage characteristics of the area. It is readily seen that determining flows for a storm or combination of storms having numerous rainfall periods will require many arithmetical operations.

Since the number of rainfall periods and the number of ordinates (in the program called distribution coefficients) vary over wide ranges an efficient program must make provision for these variations. The program presently being used requires 6 pinboards. This program will suffice for any number of distribution coefficients from 3 to 30 and for an unlimited number of rainfall periods. The printout of the solution is in the format shown in Plate XXII. The first entries are the constants used in the computations, followed by the distribution coefficients. The rainfall amounts are then entered in turn, the computer automatically printing the time period and computed flow after each rainfall entry. Plates XXIII, XXIV and XXV are "Operators Instruction," "Program Sheet" and "Memory Map." Time of solution of the sample problem, including entry of constants and coefficients was 6 minutes, as compared with 1-1/2 hours using a desk calculator.

2. Statistical Computations

Many types of statistical computations have been programmed for the E-101. One type of computation, the determination of mean, standard deviation and coefficient of skew for stream flows, is a common operation in Corps of Engineer Offices. Correlation analysis and determination of regression equations are other types of statistical operations currently experienced. These and many more are very capably solved by the E-101. A typical type of computation is the fitting of curves to experimental data. The program may be considered in two parts, the first the determination of the normal equations and the second the solution of these equations. Data (values of x and y) are entered by keyboard and at the completion of entries the normal equations are printed out. The coefficients of the equations are

1515-30 WW 1 January, 1958

50 Round-off constant
01 Count number

Dist. Coefs.	Time Period	Rainfall
00	01	00
56	02	14
429	03	51
1011	04	126
1265	05	59
1023	06	32
624	07	00
359	08	00
223	09	00
159	10	26
116	11	48
86	12	107
64	13	134
48	14	73
32	15	26
21	16	00
12	17	00
05	18	00
01	19	00
00	20	00
00	21	00
	22	00
	23	00
	24	00
	25	00
	26	00
	27	00
	28	00
	29	00
	30	00
	31	00
	32	00

	Discharge c.f.s.
00	3557
08	2512
89	1588
431	992
1266	659
2333	467
2937	341
2727	251
2009	181
1298	124
928	79
1047	46
1712	22
2838	08
3887	02
4177	00

OPERATING INSTRUCTIONS SHEET

BURROUGHS E101

September 1954

PROBLEM	PROGRAMMED BY
Unit Hydrograph Computation	E. A. LAWLER

SET-UP	PINBOARDS	PRINTING SCHEDULE	SPECIAL INSTRUCTIONS	START BUTTON
	I - VI	I	Set "Y" from table below	1

SIGNAL (K, H)	PINBOARD AND STEP		PINBOARD CHANGES, MANUAL INSTRUCTIONS, ETC.	START BUTTON	KEYBOARD ENTRY X	KEYBOARD ENTRY Y	KEYBOARD ENTRY NUMBER	MOTOR BAR
K	1	0	Enter Roundoff constant				50	3
K	1	2	Enter Count number				14	3
K	2	2	Enter Coefficients followed					3
			by zeros (B key) until "Halt"					
H	2	13	Clear and Press Start Button 3					
K	3	4	Enter rainfall amounts;					2
			after last rainfall amount enter					
			zeros (B key) until zero					
			appears in discharge column					

No. of Coefs.	Set Y
4, 5, 6	1
7, 8, 9	2
10, 11, 12	3
13, 14, 15	4
16, 17, 18	5
19, 20, 21	6
22, 23, 24	7
25, 26, 27	8
28, 29, 30	9

PLATE XXIII

PLATE XXIV

Problem: Unit Hydrograph Computation E101 MEMORY MAP Page No.:

Programmer: L. G. Fawcett

	0	1	2	3	4	5	6	7	8	9
0	X_0	Y_0	Y_1							X_{10}
1	X_{11}	X_{12}								X_{20}
2	X_{21}	Y_{22}	Y_{23}							X_{30}
3										
4	$\Sigma A_{i3} X_{i0}$	$\Sigma A_{i3} X_{i1}$	-						-	$\Sigma A_{i3} X_{i0}$
5	$\Sigma A_{i9} X_{i1}$	-	-	-	-			-	-	$\Sigma A_{i9} X_{i0}$
6	$\Sigma A_{i9} X_{i1}$	$\Sigma A_{i9} X_{i2}$	-	-	-			-	-	$\Sigma A_{i9} X_{30}$
7										
8										
9	.50 Runoff		1 Count	Σ time periods					$\Sigma A_{i9} X_{i1}$	$\Sigma A_{i9} X_{i1}$

PLATE XXV

re-entered and the final curve coefficients are then printed out. To illustrate the procedure a second order curve is computed from the basic data shown in Plate XXVI. The coefficients for the normal equations

$$AN + b\Sigma(X) + C\Sigma(X^2) = \Sigma(Y)$$

$$A\Sigma(X) + b\Sigma(X^2) + C\Sigma(X^3) = \Sigma(XY)$$

$$A\Sigma(X^2) + b\Sigma(X^3) + C\Sigma(X^4) = \Sigma(X^2Y)$$

are shown in the print out. Re-entry of these coefficients after transposition of right term of equation completes the operation - the coefficients a, b and c of the equation

$$Y = C + b_x + ax^2$$

being print out as shown.

It should be pointed out that the program could be written to eliminate the re-entry of the normal equations for simultaneous solution. However, to obtain greater accuracy for the final curve a normalizing procedure is followed. For purposes of discussion assume the decimal point to be seven places from the right, making the first row of values read 7.0, 21.0, 91.0, and 3.49. The second row would be 21.0, 91.0, 441.0 and 9.105, and the third row 91.0, 441.0, 2275.0 and 38.39. Since it is permissable to divide all values in any row by any constant it is feasible to divide the second row by 10 and the third row by 100. Further normalizing can be employed by dividing all values in any column by a constant and then applying that constant to the final coefficient. In this instance the latter procedure does not appear to be indicated. Re-entry of the normalizied coefficients by keyboard entry accomplishes the final computations and the coefficients for the equation

$$y = c + bx + ax^2$$

as printed out are

$$y = .717370 - .135818x + .014512x^2$$

Plates XXVII to XXIX are the "Program Sheet," "Memory Map" and "Operator Instruction."

Programs are available for a very large number of problems. These are available from the manufactures and it would appear to serve no purpose to list them.

REFERENCES

1. Thomas, H. A. "The Hydraulics of Flood Wave Movements in Rivers." Carnegie Institute of Technology, Engineering Bulletin, 1934.

2. "Flood Control," Chapter V "Flood Routing," The Engineering School, Ft. Belvoir, Va. 1946.

3. Sherman, L. K. "Stream Flow from Rainfall by the Unit Graph Method," Engineering News Record, Vol 108, 1932, pp 501-505.

4. Snyder, F. F. "Synthetic Unit Graphs," Transactions, A.G.U., Vol. 19, Part I, 1938, pp 447-454.

10000000 constant

X's	Y's		
00000000	7370000		
10000000	5720000		
20000000	4900000		
30000000	4450000		
40000000	4220000		
50000000	4140000		
60000000	4100000		
a's	b's	c's	d's
70000000	210000000	910000000	34900000
210000000	910000000	4410000000	91050000
910000000	4410000000	22750000000	383990000

Rationalized

a's	b's	c's	d's
70000000	210000000	910000000	34900000-
21000000	91000000	441000000	9105000-
9100000	44100000	227500000	3839900-
717370128	135317828-	14511904	
a	b	c	

PLATE XXVI

BURROUGHS E101 Program Sheet	Problem: Curve Fitting - Second Order Curve	Programmer: E. G. Lawler	Date: Sept. 1956	Page No.

PB I

0	H00		
1	K	-W	Enter a_1
2	A5		
3	B		
4	K	-W	Enter b_1, c_1, d_1
5	A22		
6	÷E0		
7	S02		
8	U04		
9	H0F		
10	R00		
11	-00		
12	WE9		
13	S02		
14	U012		
15	U24		

PB II

0	RXF		
1	P2		
2	S1Y		
3	U00		
4	H10		
5	K	-W	Enter a_2
6	B		
7	H0Y		
8	XEF		
9	+E9		
10	WE9		
11	S02		
12	U08		
13	S1Y		
14	U05		
15	U32		

PB III

0	H10		
1	U10		
2	H0Y		
3	K	-W	Enter b_2
4	A22		
5	+E9		
6	WE9		
7	S02		
8	U03		
9	RX9		
10	-X9		
11	-X9		
12	A12		
13	WX9		
14	H0F		
15	U40		

PB IV

0	RX9		
1	B		
2	RE9		
3	÷EF		
4	REF		
5	B		
6	H10		
7	XXF		
8	A12		
9	+EF		
10	WEF		
11	S1Y		
12	U07		
13	302		
14	U00		
15	U19		

PB V

0	R09		
1	-00		
2	H00		
3	H10		
4	WEF		
5	S19		
6	U04		
7	K	-W	Enter 1
8	W90		
9	K	-W	Enter X_1
10	W91		
11	B		
12	X91	-8	X_1^2
13	A1W	-W	
14	W92		
15	U60		

PB VI

0	X92	-8	Y_1^2
1	A1W	-W	
2	W93		
3	X93	-8	X_1^3
4	A1W	-W	
5	W94		
6	K	-W	Enter Y_1
7	W95		
8	B		
9	X91	-8	$X_1 Y_1$
10	A1W	-W	
11	W96		
12	X92	-8	$X_1^2 Y_1$
13	A14	-W	
14	W97		
15	U70		

PB VII

0	H10		
1	R9F		
2	+0F		
3	W0F		
4	317		
5	U01		
6	U59		
7	P1	-W	$\Sigma(XY)$
8	R02		
9	P2	-W	$\Sigma(X^2)$
10	R03		
11	P2	-W	$\Sigma(X^3)$
12	R04		
13	P2	-W	$\Sigma(X^4)$
14	R07		
15	P1#	-W	$\Sigma(X^2Y)$

PB VIII

0	R00		
1	P2	-W	N
2	R01		
3	P2	-W	$\Sigma(X)$
4	R02		
5	P2	-W	$\Sigma(X^2)$
6	R05		
7	P1	-W	$\Sigma(Y)$
8	R01		
9	P2	-W	$\Sigma(X)$
10	R02		
11	P2	-W	$\Sigma(X^2)$
12	R03		
13	P2	-W	$\Sigma(X^3)$
14	R06		
15	U77		

PLATE XXXIII

Form P.S. 1-56 2M

Problem Curve Fitting - Second Order Normal Equations
E101 MEMORY MAP
P b Y
Programmer: E. O. Kaufman
Page No: 1
0 1 2 3 4 5 6 7 8 9
Σ 1 Σ x Σ x² Σ x³ Σ x⁴ Σ y Σ xy Σ x²y
0 1 2 3 4 5 6 7 8 9
N

PLATE XXVIII

OPERATING INSTRUCTIONS SHEET

BURROUGHS E101

PROBLEM: Curve Fitting - Second Order Curve $Y = a + bx + cx^2$ — PROGRAMMED BY: E. A. Lawler

SET-UP	PINBOARDS	PRINTING SCHEDULE	SPECIAL INSTRUCTIONS	START BUTTON
	I - VIII	-	-	5

SIGNAL (K, H)	PINBOARD AND STEP		PINBOARD CHANGES, MANUAL INSTRUCTIONS, ETC.	START BUTTON	KEYBOARD ENTRY X	KEYBOARD ENTRY Y	KEYBOARD ENTRY NUMBER	MOTOR BAR
K	5	7	Enter Constant				1	1
K	5	9	Enter $X_1, X_2, \ldots X_n$					2
K	6	6	Enter $Y_1, Y_2 \ldots Y_n$					1
K	5	9	Clear and Print out	8				
			- - - - - - - - - - - - - - -					
			Start Button 1					
K	1	1	Enter a_1		-	-		
K	1	4	Enter b_1, c_1, d_1		-	-		
K	2	5	Enter a_2		0	0		
K	3	3	Enter b_2, c_2, d_2		0	0		
K	2	5	Enter a_3		1	1		
K	3	3	Enter b_3, c_3, d_3		1	1		
K	2	5	Clear and Print out	2	2	2		

PLATE XXIX

Paper 1516 January, 1958 WW 1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CLOSURE OF THE BREACH IN BAYOCEAN PENINSULA, OREGON[a]

Harlan E. Brown, Jr.,[1] A.M. ASCE, Gerald D. Clark,[2] A.M. ASCE, and Robert J. Pope[3]
(Proc. Paper 1516)

ABSTRACT

"Closure of the Breach in Bayocean Peninsula, Oregon," by Harlan E. Brown, Jr., Gerald D. Clark and Robert J. Pope. Bayocean Peninsula, a natural sand, gravel and boulder barrier about 4 miles in length which separated Tillamook Bay from the Pacific Ocean, was breached by ocean storm waves. This paper presents the history of erosion and the design and construction of rock fill and sand fill closure structures.

Bayocean Peninsula is a natural sand, gravel, and boulder barrier about 4 miles in length, separating Tillamook Bay from the Pacific Ocean. In November, 1952, the narrow portion of the peninsula was breached by ocean storm waves following many years of beach erosion. The closure structure described herein was designed and located to assist the forces of nature in rebuilding a barrier to protect the bay, thereby preserving its usefulness.

Figure 1 is a map of Tillamook Bay and vicinity, and Figure 2 is an aerial view of the Bayocean Peninsula taken in May 1939. Bayocean Peninsula is located on the Oregon Coast about 60 miles west of Portland and 47 miles south of the mouth of Columbia River. The Peninsula derives its name from

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1516 is part of the copyrighted Journal of the Waterways Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

a. Paper presented at a meeting of the American Society of Civil Engineers, New York, N. Y., October, 1957.

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3. Chief, Soils Engineering Section, Portland District, Corps of Engineers, U. S. Dept. of the Army, Portland, Ore.

the old resort town of Bayocean, which once flourished near the midpoint of the peninsula.

Tillamook Bay is 6 miles long in a north and south direction and has a width of 3 miles. Its area at high and low waters are 14 and 7-3/4 square miles, respectively, with a tidal prism of about 58,000 acre-feet. Five rivers: the Miami, Kilchis, Wilson, Trask, and Tillamook, with total drainage area of 573 square miles, flow into the bay.

Prior to November 1952, Tillamook Bay was connected to the Pacific Ocean by the channel around the north end of Bayocean Peninsula. The entrance was improved during the years 1913-1917 by the construction of a north jetty extending 5,400 feet seaward from the coast line. The constructed crest height of the jetty above mean lower low water was 16 feet. The jetty was constructed to assist in maintaining a navigation channel 18 feet deep at mean lower low water from the ocean to the mill town of Garibaldi at the northeast corner of the bay. In 1933, the jetty was rehabilitated and extended an additional 300 feet. Tidal range at the entrance, from mean lower low to mean higher high water, is 7-1/2 feet, and the extreme range is about 13-1/2 feet. In the past, dredging on the ocean bar has been necessary only at intervals of several years inasmuch as the project depth of 18 feet has been fairly well maintained by the natural scouring action of the ebb tides, aided by the north jetty.

Bays and estuaries along the Oregon coast were formed by submergence. Whether the submergence was caused by the coastwise sinking of the land or the return to the sea of water locked up in the continental ice sheets, the result was the forming of bays and estuaries along the lower stretches of rivers. Subsequent erosion and straightening of the shore line formed sand spits and barrier bars. Bayocean Peninsula resulted in part from a northward movement along the shore of heavy debris and basalts from Cape Meares, 5 miles south of the bay entrance. Beach sand undoubtedly is moved in the same direction by the shore drift during the southwest storms of winter. Wind action from the northwest has also played an important part in the building of the peninsula, especially that part above the elevation of the beach gravel.

Prior to November 1952, the peninsula varied in width between high-water lines from 300 feet near the southerly end to 3,300 at a point 1 mile from the northerly end. Elevations varied from 17 feet above mean lower low water, on a sand, gravel, and boulder formation at the south end, to 140 feet on the highest dunes located near the middle of the peninsula. The northerly portion of the peninsula consists of sandy barrier beach nearly 1 mile long, with an average elevation of 15 feet and with occasional dunes rising to elevations of nearly 60 feet. The southern 1-1/2 miles formerly consisted of a continuous ridge of relatively uniform elevation except for three low gaps. The gaps, known as South, Jackson, and Natatorium Gaps, were located at the southerly end of the peninsula, near the center of the narrow spit, and just south of the resort town of Bayocean. The gaps had lengths, between high-water lines on the ocean and bay sides, of approximately 1,300, 400, and 1,200 feet, respectively. Infrequent storms of unusual severity washed through all three gaps, causing severe erosion along the ocean front. A storm in 1939 cut the road and water line serving the residents of Bayocean.

After completion of the Tillamook Bay jetty, a large volume of sand was impounded on the north side to an average elevation well above high water. The accretion extended a distance of over 2 miles to the north, and

developed a stabilized width at the jetty of about 3,000 feet.

Statements of residents of Bayocean are not consistent as to when erosion was first noticed along the peninsula. At the time the resort town of Bayocean was first laid out, the seaward side of the barrier dune consisted of a gradual slope with vegetation down to the lines of the usual backshore scarp. Erosion of the slope apparently was not noticeable, or was of such seemingly inconsequential and local nature that it did not become a matter of concern until subsequent to 1930, and its rate of progress was not recognized as a menace until 1937. Sufficient survey information of the shore and water areas in front of the peninsula is not available to determine when erosion first started.

Borings along the lines of Jackson and Natatorium Gaps indicate that the peninsula rests largely upon lagoon deposits that were probably laid down when the peninsula was farther seaward, and when the bay occupied the area now covered by the high dunes. At extreme low tide, large mats of submerged peat have been found seaward of the peninsula. Such evidence strengthens the theory that the barrier spit has migrated landward in recent geological times.

Erosion of the seaward bank of the peninsula, measured from the front line of the lots as originally plotted in 1907 to August 1938, varied from 40 feet at the south end of the peninsula to 200 feet near the northern end. During the last few years, prior to construction of the breakwater, the rate of erosion appears to have accelerated with the passing of each year.

Comparison of cross sections made in 1939 and 1946, near the midpoint of the peninsula, indicates erosion above elevation +20 at the rate of about 20 feet per year during that period. The resort town of Bayocean was a constant victim of erosion. Houses were undermined and washed to sea, and other houses were moved to avoid destruction.

Observation and geomorphology indicate that the Oregon coast line is undergoing a recessional change and that, without the influence of man-made structures, erosion would be the normal expectancy at any point along the coast. Erosion of the dune fronting Bayocean Peninsula has been caused chiefly by wave action, which in turn has been able to attack the dune because of erosion offshore which permitted large waves to approach nearer and make a stronger attack on the dune.

Other factors which may have contributed to the shore line erosion along the peninsula are the results of civilization. These include the removal of gravel from the beach front for construction of roads and dwellings, the reduction in the volume of driftwood along the beach through its removal as firewood, and the virtual depletion of its supply with the discontinuance of logging operations adjacent to tributary coast streams.

During the month of November 1952, severe storm waves, in combination with higher than ordinary tides, breached the narrow southern portion of the peninsula. Succeeding high tides and storms continued to widen and deepen the newly-created entrance. Within a short time, the gap was nearly a mile wide, and the bay and ocean were connected at all but the lower stages of the tide.

With the loss of protection against wind and wave action formerly afforded by the peninsula, the entire bay area was subjected to considerable damage. During high tide, waves rolled through the break and spread out in a fan-shaped pattern. This action produced a troublesome surge action at both the north and south ends of the bay. At the north end, operators of log-booming grounds had several log rafts broken up each winter. At the south end,

levees along the lower reaches of the tributary rivers were weakened by the additional wave wash and threatened with complete failure or overtopping by salt water due to the increased tidal range and surge effect. Sand carried in by storm waves soon covered over one-third of the approximately 3,000 acres of tidelands leased for oyster beds. A natural channel along the east side of the bay was shoaled to such an extent that rafting of logs could be accomplished only during the higher of the two high tides each day. The major portion of the tidal flow entered and departed through the new entrance. With the loss of scouring current action through the improved entrance, the remaining "island" portion of the peninsula spread to the north and northwest, threatening to choke off the Federal navigation channel along the north side of the bay.

In compliance with Congressional resolution dated June 20, 1952, the Corps of Engineers reviewed the existing project in the light of changed conditions and found that benefits that would accrue to the public and to local interests were sufficient to justify closing the breach in the peninsula. The plan of improvement consisted of a dredged-sand breakwater 1.4 miles long with a top elevation of +20, a top width of 200 feet, and with seaward and bayward slopes of 1 on 20 and 1 on 10, respectively. The alinement recommended was a straight line between Pitcher Point and Bayocean Peninsula near the town of Bayocean.

Hydrographic surveys made in the fall of 1955 revealed that the breach had deepened considerably. Also, at each end of the breach, a sand and gravel spit curved bayward in a one-quarter bend and extended into the bay a distance of over one-half mile. At low tide, both spits averaged about 1,000 feet in width, and had maximum elevations of 12 to 13 feet above mean lower low water.

Final design studies and preparation of contract plans was begun in the fall of 1955 with the intention of awarding the contract in the spring of 1956. It was realized that the closure of the breach must be completed in one working season. A partially completed breakwater would be very severely damaged by the winter storms. Due to the deepening and widening of the breach and accelerated erosion of the Bayocean Peninsula, a number of modifications of the preliminary plan were studied before a design was selected on which the contract plan would be based.

The alinement for the breakwater was set well back from the beach line of the Bayocean Peninsula. It was anticipated that the pocket in the coast line formed by this setback would be filled, after completion of the breakwater, chiefly by erosion of material from the Bayocean Peninsula, and to a lesser extent by material moving northward from Cape Meares. With the pocket filled, a more or less uniform coast line would be re-established forming a protective sand beach in front of the breakwater. The northern end of the breakwater was located at a point on the eastern side of Bayocean Peninsula which, after removal of sufficient sand for filling the pocket in front of the breakwater, plus the normal rate of recession which prevailed before the breakthrough, would provide a project life of 50 years. Detailed studies led to selection of the alinement shown on Figure 4 as the most economical and suitable location meeting the above criteria.

A total of 79 borings to an average depth of 30 feet were made in the foundation area and sand borrow areas during September and October of 1955. The borings were advanced by jetting down AX casing. Samples were obtained at approximately 5-foot intervals by means of a small, hand-operated

piston-type sampler. Undisturbed samples of the foundation material for strength tests were obtained from shallow test pits at the south end of the breakwater. All samples were visually classified by the field inspector and a sufficient number of samples of each type of material tested in the laboratory to verify the visual classification. Laboratory testing consisted of mechanical analyses on the sands and moisture content and Atterberg limits on plastic materials. Consolidation tests were performed on two undisturbed foundation samples of silty sand.

Borrow explorations indicated a suitable source of sand for hydraulic dredge excavation was located in the bay east of the Bayocean Peninsula. This material was finer in gradation than desirable but was the best available. An ample quantity was available above a depth of 25 feet and within 9,000 feet of the work area.

Hard, durable basalt rock in riprap size was available from quarries located at Cape Meares headland, a haul distance of from 3 to 4 miles. Some rock from 3 to 5-ton size was also available from these quarries. High quality rock for armor stone in 2 to 5-ton size was available from a quarry on the Trask River approximately 14 miles from the site, or from a quarry at Garibaldi, approximately 16 miles from the site.

A relatively soft, friable sandstone was available from the ridge alone the bay southeast of the site. This rock was not as durable as that usually used for protective work, but was considered capable of withstanding wave action for several seasons. This material could be quarried by careful blasting in large blocks at low cost and, due to a haul distance of less than one-half mile, could be placed at about 60 percent of the cost of hard rock from Cape Meares. For this reason the specifications were written to encourage the use of this sandstone formation by defining suitable stone as "fresh unweathered, moderately hard, quarry stone, which will not disintegrate rapidly under the action of water or atmosphere and weighing at least 130 pounds per dry cubic foot."

A profile of foundation conditions along the center line of the breakwater is shown on Figure 5. The foundation under the southern part of the breakwater consisted of stratified deposits of soft silt, sandy silt, silty sand and fine sand. The silty sediments were very soft having moisture contents appreciably higher than the liquid limits. The soft foundation was covered with fine beach sand at the spits on each side of the breakthrough, but was exposed in the breakthrough and south of the south spit. The soft silty strata pinch out to the north where the foundation is predominantly fine sand.

The fine sand at the north end of the breakwater provided an adequate foundation material for the type of structure planned. The sand spits on each side of the breakthrough provided a natural blanket over the soft foundation for relief of foundation stresses in these areas. However, analysis indicated toe berms were necessary to prevent overstressing the foundation where the breakwater was founded directly on the soft sediments in the breakthrough and at the sound end of the breakwater. The designed section therefore provided blankets of quarry-run rock 6 feet thick extending 50 feet bayward and seaward of the embankment.

With sand and quarried rock as the materials available, the cost of different combinations of sand and rock fill were estimated. For the purpose of these comparative estimates, the top elevation of each type of breakwater cross section south of station 50+00 was at +20, and north of that point the

elevation of the breakwater was at +15. The estimated costs of three designs: rock fill for the full length, sand fill for the full length, and rock fill south of station 50+00 and sand fill north of that point, were $1,643,000, $1,556,000, and $1,665,000 respectively. The design employing sand fill for the entire length assumed that closure of the tidal channel could be made without excessive losses of material. An average maximum velocity of 5 feet per second through the tidal channel was expected at the time closure of the channel was begun. With the coarsest sand available for the closure being medium sand, there was considerable doubt that a closure could be made with sand fill. The closure of the channel with rock dumped from a trestle was considered to be a quick and certain method. Therefore, it was decided that rock fill should be used south of station 50+00. The design employing rock fill for the entire length of the breakwater required that all of the rock be hauled from the south end as barges could not bring rock to the northern section because shallow water prevents access except at high tide. The adopted design consists of rock fill south of station 50+00 and sand fill north of that point. Typical section of this design is shown on Figure 6. In selecting this design, consideration was given to the necessity of completing the breakwater before the winter storms begin. Simultaneous construction of the southern and northern portions of the breakwater increased the possibility that the work could be completed in one summer season.

In considering the problems which a contractor would encounter in constructing this breakwater, it was recognized that the principal problem would be the closure of the tidal channel which had a depth of about 4 feet at low tide. After study and review of procedures used in construction of embankments across flowing streams with similar bed material, it was concluded that the final closure of the tidal channel should be made in the central part of the rock embankment. If the rock embankment were to be constructed by dumping from the south end, it was believed that the tidal channel would shift northward as the embankment progressed. For this reason the plans and specifications required the contractor to leave a channel 800 feet wide until the balance of the embankment was completed to elevation +12. No rock fill was to be placed north of the channel until a blanket of quarry-run stone 3 feet thick over the width of the breakwater base had been placed in the channel. This provision of the specifications required a pile treatle to construct the 3-foot thick blanket and the rock fill north of the channel. The plans showed a suggested construction trestle which was four lanes wide and designed for H-30 loading. It was not the intent of the specifications to limit the contractor in his choice of construction facility or method of providing the tidal channel. The suggested construction trestle was intentionally conservatively designed to avoid the implication that a lighter and smaller structure would be adequate.

The Invitation for Bids was issued on March 1, 1956 and bids were received on March 20, 1956. On April 1, 1956, the contract was awarded to the low bidder, General Construction Company of Portland, Oregon. The bid was $1,163,686, which was $118,034 below the Contracting Officer's estimate. The specifications required completion of the breakwater, except for planting Holland Beach grass, on or before October 31, 1956. Planting of beach grass on the sand fill was specified to be completed on December 20, 1956.

On April 22, 1956, the contractor moved a 24-inch pipeline dredge to the site and immediately began preparatory work to lay the pipeline. Placement

of sand fill started on May 4 under a three-shift operation. This work continued on a six-day week basis with only a very few stoppages of work until the sand fill section of the breakwater was completed on August 2. After cutting a channel into the western boundary of the borrow pit, the dredge then worked along that side of the area dredging to a depth of 45 feet below mean lower low water. A booster pump was not used although the maximum pumping distance was over 7,000 feet and the average about 5,000 feet. The pumped fill material was contained between high ground on the west and toe dikes on the east. The toe dikes were constructed 500 to 1,000 feet ahead of the main fill by depositing dredge material which, after draining, was raised to grade with bulldozers.

On April 9, the contractor began exploratory drilling and stripping of the Pitcher Point quarry. Drilling operations for quarry rock began on May 1. The Pitcher Point sandstone is soft and friable as compared to the usual quarry rock available in the Pacific Northwest. After ten days of experimenting with drilling and loading procedures, a successful quarry shot was made on May 10. The contractor began hauling rock to the fill on May 17. The fill was started at Pitcher Point by end dumping into the shallow channel between the mainland and the spit which formed the south shore of the tidal channel created by the breaching of the Bayocean spit. The bottom of the small bay west of the breakwater and south of the south spit, as shown on Figure 4, is all above the elevation of mean lower low water. During a tidal cycle there is a flow of water into and then out of the bay.

By June 15 the fill had been constructed 40 feet wide at a top elevation of +8 to station 4+30, which was approximately 220 feet distant from the +6 contour on the south spit. At this time placement of fill was stopped because the south sandspit was eroding. The contractor stockpiled about 300 tons of quarry-run material on the fill and then at low tide, using the stockpiled material and material trucked from the quarry, the closure to the south spit was successfully completed in about two and a half hours. This fill was 15 feet wide at elevation +4 and after the closure operations the contractor continued to raise it in advance of the rising tide. However, the contractor's only shovel broke down at this critical time and the rising tide overtopped the fill and washed 4,000 cubic yards of sand out of the sandspit north of the end of the fill. On June 18, with 1,500 tons of rock stockpiled, a second attempt was made to close to the south sandspit. This attempt started at station 5+70 and was abandoned at station 7+00 without reaching the sandspit. About 8,000 cubic yards of sand were washed out during this try. Failure of this attempt was due to poor timing of the operation. Finally on June 21, with 5,000-ton stockpile and by timing the closure to coincide with a minus 1-foot tide, the closure to the south sandspit was made and held. The difficulties encountered in making the closure of this comparatively small, shallow channel emphasized the necessity for careful planning before attempting the main closure.

The rock fill was made across the south sandspit as rapidly as possible so that trestle construction could be started. On July 3, the 800-foot long and 20-foot wide trestle was started and on August 7 it was completed. Immediately following the placing of a section of the decking, quarry-run rock was dumped off both sides of the trestle to protect the piling from scour. During this time the rockfill section south of the trestle was completed to elevation +20.

After completion of the trestle a 50-foot by 50-foot turnaround was constructed of quarry-run rock at the north end of the trestle. This was necessary as the 20-foot wide trestle was not wide enough to permit two-way truck traffic. The turnaround was completed on August 13. On August 15, with a large stockpile available and a standby shovel in the quarry, the contractor in seven hours extended the fill to effect a closure to the north sandspit. This fill was 15 feet wide at the top which was at elevation +5.5. Only minor erosion of the north sandspit occurred during this operation.

On August 21 work commenced on widening the 3-foot thick blanket, in the bottom of the tidal channel, to the required width of 110 feet. During the two daylight shifts, trucks hauled quarry-run rock which was dumped off the side of the trestle. During the night shift, a crane with a 70-foot boom and one and one-half cubic yard clamshell bucket, operating from the trestle, spread the 3-foot blanket. Only enough material was dumped off the trestle each day to keep the clamshell busy during the following night. During the balance of the day shifts, quarry-run rock was hauled to a section of rock embankment north of the trestle. This section was accessible only during periods of low tide. Soundings made once or twice a week in the tidal channel showed an average maximum scour of about 3 feet with some areas of accretion. As the scouring action of the tide in the channel was not serious, the contractor was authorized, on August 21, to complete the fill north of the trestle to elevation +12 although the 3-foot blanket had not yet been completed. This work was authorized prior to completion of the rock blanket because the scour in the channel was not serious and so that sand would build up on the seaward side of the embankment rather than wash over the partially completed rock fill into the bay. On August 27, the fill north of the trestle was completed to elevation +12 and station 50+00, the north end of the rock embankment, and on September 4 the rock blanket in the tidal channel was completed.

The next operation was the closure of the tidal channel which, during the planning of the project, had been expected to be the most difficult part of the work. Operations up to this stage had demonstrated that with careful planning and normal sea conditions, the closure could be made. Starting on September 3, the contractor constructed a windrow on the seaward side to elevation 0.0 just outside the piling of the trestle. Dumping operations were evenly distributed along the trestle so that the top of the windrow was raised uniformly. After the seaward windrow was built to elevation 0.0, a similar windrow was constructed in the same manner to elevation +4 on the bayside of the trestle. Operations were then shifted to the seaward side and that windrow raised until, on September 13, the flow of water at high tide over the windrows was completely stopped. Both windrows were then raised to elevation +14 by September 18, at which time the closure was considered complete.

After final closure, the rock fill was completed to final grade and cross section, including the 6-foot thick toe blankets. Heavy rains during the latter half of October delayed the work so that completion and acceptance of the embankment was delayed until November 16, sixteen days after the specified completion date of October 31, 1956.

Grading and leveling of wind-blown dunes on the sand fill section started on November 1. After spreading fertilizer on the sand fill, planting of Holland Beach grass began on November 8. This work, which completed the contract, was finished December 22, 1956. An aerial view of the completed job is shown on Figure 9.

The total payment to the contractor at completion of the project was $1,140,736.23 for the following quantities of work:

Item	Unit	Quantity	Amount
Class A Rock	Ton	26,199.95	$ 55,019.90
Quarry-run Rock	Ton	323,094.11	567,603.05
Sand Fill	C.Y.	1,935,546.00	406,464.66
Beach Grass	Acre	104.9	28,648.62
Misc. Contract Items			83,000.00
			$1,140,736.23

Immediately after completing the closure, commercial fishermen reported marked increases in tidal velocities in the navigation channel and rapid erosion of sandbars which had been building inside the bay since 1952. Sanding-in of the embayment seaward of the breakwater has progressed as is illustrated by Figure 10 showing conditions in October 1956 and March 1957. During the winter months storm-water runoff washed a channel adjacent to the breakwater from a lake formed at the south of the breakwater to the embayment. This caused some undercutting of the breakwater, but more serious, tidal action between the lake and embayment kept the channel open and prevented the desired sanding-in of the embayment. This channel was temporarily closed in June with a sand and drift log fill and an outlet cut through the barrier bar between the ocean and the lake. The embayment is now sanded in solid for a distance of 300 to 400 feet seaward from the breakwater. From observations over the past summer it appears that the complete sanding-in of the embankment cannot be accomplished as long as the drainage from the lake is along the breakwater, thence seaward through the embayment. The temporary drainage from the lake through the barrier bar to the ocean was experimental and accomplished at very little cost. Should the above method of drainage prove unsatisfactory, there is the possibility that it may be necessary to force the drainage to pass from the lake through the breakwater and into Tillamook Bay.

Only a minor amount of repairs in the form of additional revetment on the bayward side has been necessary to date, October 1957.

CONCLUSIONS

Careful studies developed an economical design for the Bayocean breakwater which has been successfully constructed in one summer working season. Construction of the breakwater will materially reduce maintenance dredging of the entrance to Tillamook Bay and will also stop the accretion of sand on the remaining oyster beds in the south part of the bay. Performance of the breakwater in rebuilding a sand barrier to preserve its usefulness has progressed satisfactorily to date. Although continued slow erosion of the peninsula is expected, it is anticipated that the project will have a useful life of 50 years with nominal maintenance.

The Bayocean breakwater was designed and constructed under the direction of Colonel Jackson Graham, District Engineer, U. S. Army Engineer District, Portland, Oregon, Corps of Engineers. The project was constructed by the General Construction Company of Portland, Oregon.

TILLAMOOK BAY, OREGON

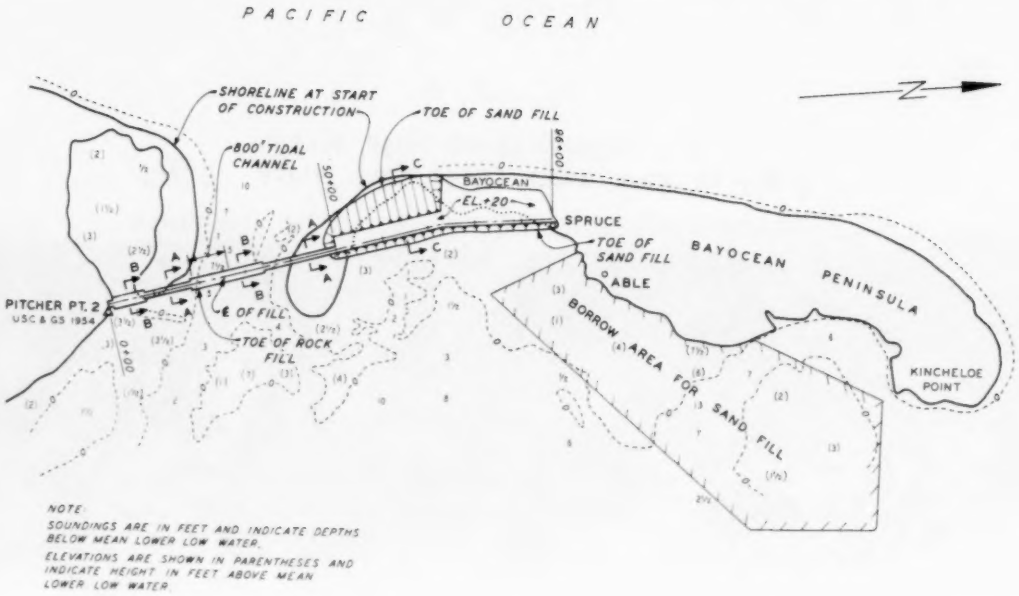
SCALE IN MILES

1 0 1 2 3

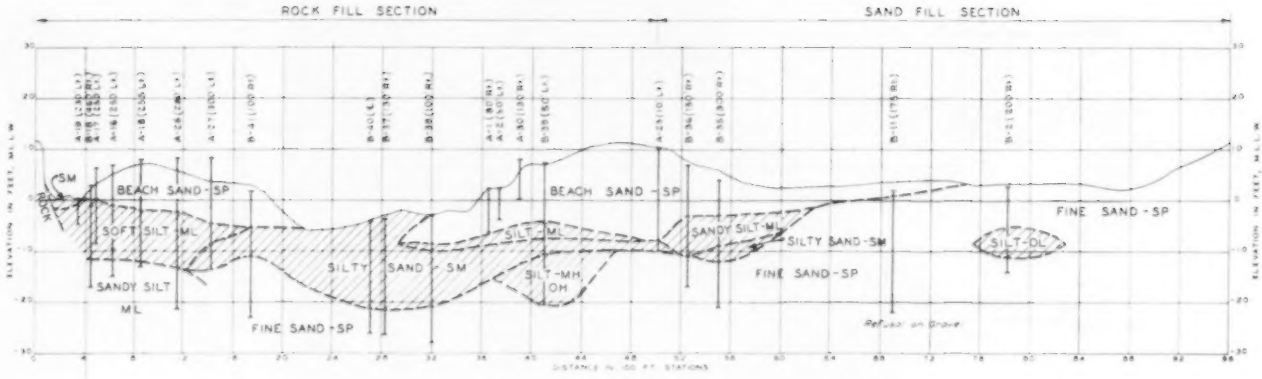
Fig. 1

Fig. 2 Aerial view shows Bayocean Spit May 1939 before break through.

Fig. 3 Aerial view shows same area as Fig. 2 in 1956 after break through.



PLAN
BAYOCEAN BREAKWATER
Fig. 4

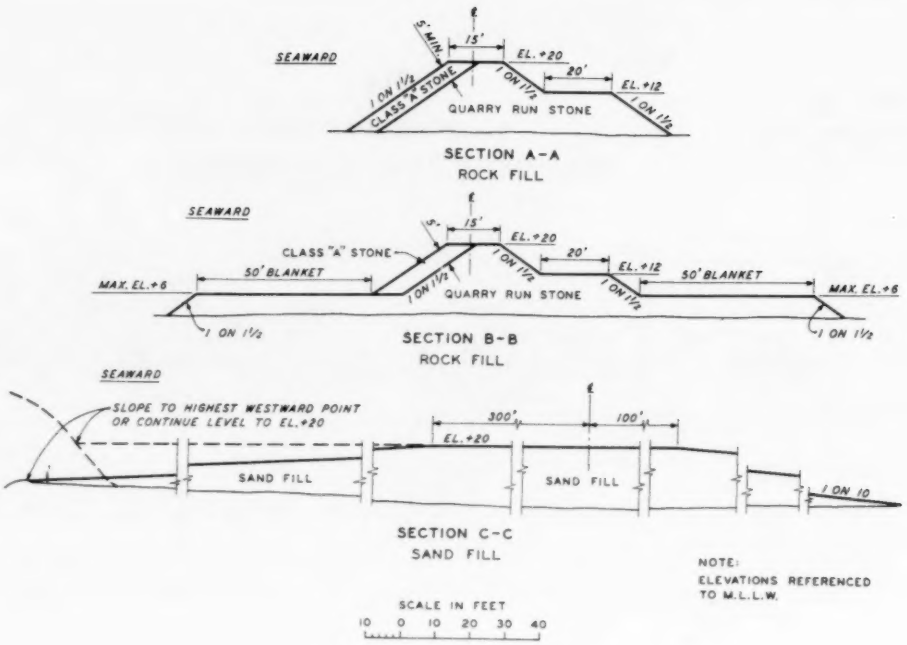


BAYOCEAN BREAKWATER

SOIL PROFILE - ℄

Fig. 5

NOTE:
HATCHING INDICATES
SOFT SEDIMENTS.



BAYOCEAN BREAKWATER
TYPICAL SECTIONS

Fig. 6

Fig. 7 Quarried sandstone being dumped for breakwater.

Fig. 8 Partial closure of tidal channel

Fig. 9 Aerial view of breakwater at completion, Oct. 1956

OCTOBER 1, 1956

MARCH 26, 1957

BAYOCEAN CLOSURE

Fig. 10 Aerial views showing sanding in of embayment seaward of breakwater between Oct. 1956 and Mar. 1957

Paper 1517 January, 1958 WW 1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

FLOOD CONTROL IN NEW ENGLAND[a]

Alden K. Sibley[1]
(Proc. Paper 1517)

SYNOPSIS

This paper discusses the problems of river and coastal floods in New England and the methods used by the Corps of Engineers to control these floods.

The New England floods of August and October 1955 took more than 100 lives and caused damage estimated at $580 million. Climaxing a long history of flooding, the 1955 disasters aroused a public demand for protection. Congress, in response to these demands, instructed the Corps of Engineers to review the flood control plans for all rivers of the Northeast and appropriated funds to begin construction of several badly needed flood control works already authorized.

Control of another type of flooding, that induced by hurricane tidal surges, is also under study. Within a period of six weeks in the fall of 1954, three hurricanes struck the coast of New England and the worst of them, hurricane "Carol" of August 31, left in its wake 60 dead and losses of approximately $300 million. Study of this problem has led to development of preliminary plans for tidal flood-protection structures that are unprecedented in concept, function, and design.

INTRODUCTION

The Corps of Engineers, long active in the construction of navigation improvements on rivers and harbors, entered the field of flood control in New

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1517 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

a. Paper presented at a Meeting of the American Society of Civil Engineers, New York, N. Y., October 15, 1957.

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England following the flood of 1927. The so-called "308" reports, prepared at the direction of Congress,[3] contained results of the first Federal investigations of flood control, water power, conservation storage, and allied water resources.

In the ensuing 30 years, the Corps planned, constructed and operated flood control projects in New England, most of which followed the major floods of 1936 and 1938. During World War II only a few localized floods occurred, and public interest in flood control waned. In the years following World War II, the New England area remained woefully unprepared for the onslaught of the 1955 floods.

Only nine flood control reservoirs and fourteen local protective works were constructed before 1955, somewhat less than twenty percent of the "comprehensive" program recommended by the Corps for New England. During the flood of August 1955, only those few cities and towns defended by completed reservoirs, dikes, and floodwalls escaped serious damage.

Since the 1955 flood three local protection projects have been completed, 13 reservoirs and 5 local works are under construction or in advance design, and 15 reservoirs and 3 local works are authorized.

In addition to river flood control, protection from tidal damage of such heavily built-up areas as Narragansett Bay and New Bedford Harbor was recommended this year.

The first part of this paper will be devoted to the orthodox problem of river flood control in five of the major river basins of New England: the Housatonic-Naugatuck, Connecticut, Thames, Blackstone and Merrimack. The second part deals with the relatively new field of hurricane tidal flooding and design criteria for tidal barriers along the New England coast.

PART I

The Flood Problem in New England River Basins

Hydrology and Economics of New England River Basins

The typical New England river basin flood exhibits certain characteristics which distinguish it from floods in other parts of the world. Whether one considers the heavily groined and revetted channel of the Oder in Germany; the wide, depressed flood plain of the Yangtze Kiang in China; or the drainage systems of the great plains and the American Southwest, one rarely encounters the peculiar combination of hydrologic and economic factors characteristic of the river basins of Northeastern United States.

Geologically, New England is a hilly, glaciated region of stony uplands cut by narrow, irregular drainage patterns of numerous small streams, lakes, and ponds. In three hundred years its industrial growth has been confined to a strip of coastline and the narrow flood plains of its drainage basins. Five of these basins are considered principal flood producers (Plate No. 1): the Housatonic-Naugatuck in the west, the Connecticut extending from Canada to Long Island Sound, the French-Quinebaug-Thames system debouching at New London, Connecticut, the Blackstone feeding Narragansett Bay at Providence, Rhode Island, and the Merrimack flowing through New Hampshire and north-

3. Authorized by the River and Harbor Act of 1927.

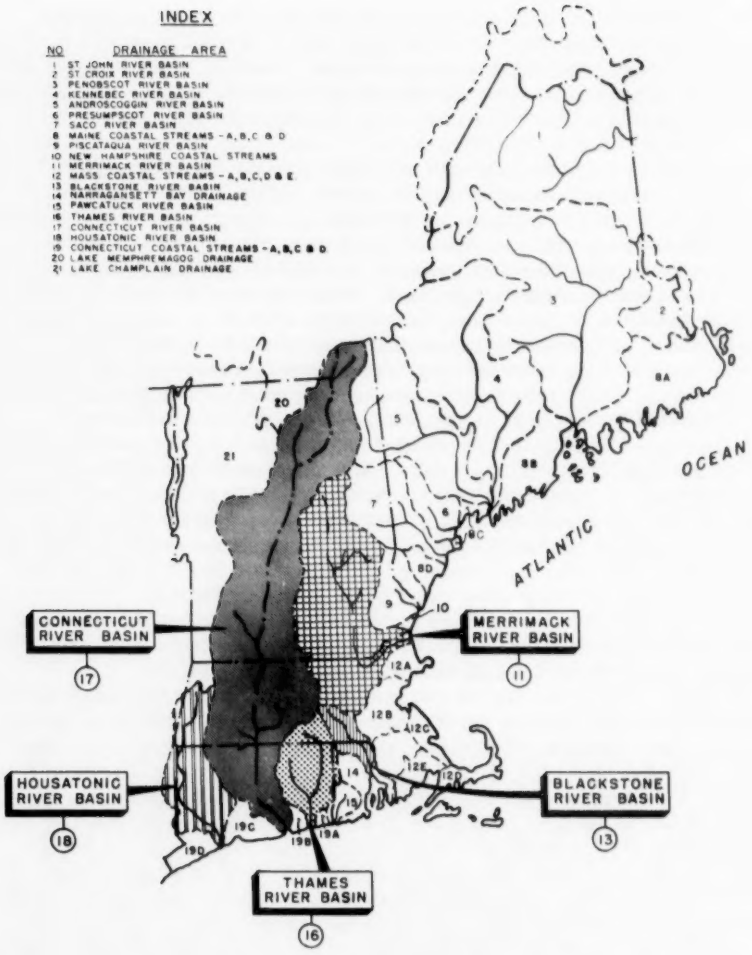
eastern Massachusetts into the Atlantic. A series of drainage basins less subject to damaging floods extends from the Androscoggin and the Kennebec in southern Maine to the valley of the St. Croix emptying into Passamaquoddy Bay at the Canadian border.

During the first two centuries of our history, New England was, and in many respects remains today, the industrial center of the United States. Forty percent of our finished brass and bronze production, for example, is in the tiny Naugatuck Valley. And because of the rocky and hilly topography, the narrow flood plains of these five basins are today practically continuous ribbons of industrial and population concentration. Flood vulnerability has increased roughly exponentially with economic growth. Significant of this accelerating vulnerability in more than three hundred years of record is the fact that the greatest flood losses but probably not the largest floods have occurred most recently. Of the cumulative billion and a third dollar flood loss of record over the past 30 years, nearly fifty percent, or $530 million, was suffered in the last flood in New England. This industrial concentration in the narrow flood plains yields the first principle of New England flood control planning: direct protection of the major damage centers by construction of large dams on the main river stems is rarely justified economically.

The second principle has a similar economic basis. The tributary systems of the five major flood basins are generally steep, narrow and "flashy." These tributary valleys, set in the rugged and hilly terrain, have become densely populated along the river banks. Any adequate flood control plan must be designed to protect not only the economies of the main river valleys but those of the tributaries as well. Large structures like the John Martin and Denison Dams on the main stem of the Arkansas, for example, if built on the Connecticut above Hartford would not only flood out Springfield, Massachusetts, but would afford no protection to the tributary valleys. The facts thus point to a large number of dams on the tributaries rather than to a few dams on the main river stems as part of the solution to the flood control problem in New England; but only part of the solution.

Hydro plants, the paper, textile and metals industries and municipal water supply have developed and are saturating progressively the tributary valleys with reservoirs. Suitable flood control sites combining adequate reservoir capacity with substantial watershed control are ever fewer and more costly to develop. The Deerfield River alone, a major flood producing tributary of the Connecticut has been so completely developed for hydro power by the New England Electric System that scarcely a foot of head is left for flood storage. By the same token, the upper Connecticut itself has been described as one of the hardest working rivers in the world, with the tail race of practically every dam discharging into the impounded waters of the power storage below. From this dearth of reservoir sites on the tributaries, the third and fourth principles of New England flood control are evident: multiple purpose reservoirs for power and flood control are usually uneconomical; and those few tributary sites which remain suitable for flood control must be supplemented by local protection works: channel improvements, dikes, flood walls, diversions and the like, if adequate protection of the main stream damage centers is to be achieved.

From these four principles, the problem of New England flood control design begins to take shape. In broad outline it involves an integrated flood control system for each of the five major flood basins, each system designed



MAJOR FLOOD PROBLEM AREAS
NEW ENGLAND

PLATE NO. 1

to be operated in time of flood as a coordinated whole. Meteorologically however, this poses a fifth and very delicate problem for the hydrologist. Though densely populated and highly developed economically, the entire area of the six New England states is smaller than a single large drainage basin in the Middle West. The Fort Peck reservoir on the Missouri, for example, is more than one-third of the area of the entire State of Rhode Island. Thus the wide expanse of precipitation from tropical hurricanes to which New England is subject, normally affects not only the entire watershed of a given drainage basin but frequently several basins simultaneously. Tributary runoff is therefore closely synchronized, each tributary contributing concurrently to flood peaks in the main channel downstream. The hydrologist's problem is thus to design a reservoir system to desynchronize simultaneous tributary contributions to the downstream flood crest and to calculate the capacities of local protective works at the main damage centers adequate to contain this reduced flood crest. This presupposes centralized control and the most delicately coordinated operation of the tributary reservoirs.

The hydrologist is further challenged by a sixth distinguishing characteristic of New England floods: they are torrential in nature; not floods of inundation. The steep slopes and narrow valleys, particularly under conditions of soil saturation from continued light rainfall, have somewhat the hydrological characteristics of a tin roof. Entire lumber yards, freight cars, and even boulders picked up and entrained by high velocity runoff attain momenta which produce the effects of battering rams against the densely concentrated structures downstream. Man-made flash floods resulting from errors in judgment in the operation of a tributary reservoir system in the Naugatuck for example could contribute to the havoc. To minimize this danger from "human error" in time of flood, numerous hypothetical examples of flood routing must be computed by the hydrologist. Observed isohyetal charts of actual storm precipitation are transposed geographically to produce a series of "type floods" which could occur in each of the drainage basins. By routing these hypothetical floods through the reservoir systems, rules for operation of the individual reservoirs under varying conditions are developed with a view to making reservoir operations as nearly foolproof as possible.

The seventh and final characteristic of New England floods is the fact that they are aseasonal in nature. Tropical hurricanes, variable rainfall characteristics, snow and ice melt runoff and winter thaws combine to produce a twelve-month flood cycle for which experience curves indicate damaging floods to be probable any month in the year. In contrast to the Sierra streams like the Truckee River in which spring runoff from snowmelt is an essential component of a damaging flood, in New England the months of January, April, August, October and December have all witnessed damaging floods of record. Because of their flashiness and unpredictability, operation of the flood control warning system in New England is a "round the clock" twelve months a year business.

From this summary analysis of the hydrology and economics of New England river basins, one may conclude that some seven distinguishing characteristics of New England river floods combine to differentiate the flood control problem in that area from the problem in other parts of the world. These seven characteristics in combination give rise to a special set of principles on which river flood control planning in New England must be based:

(1) Large dams on main stems are economically prohibitive.
(2) Watershed control must be achieved by numerous reservoirs on the tributaries.
(3) Power and flood control multiple purpose projects are rarely justified.
(4) In each river basin, local protective works must supplement tributary reservoirs to form an integrated system for the basin.
(5) Reservoir systems must be designed to permit particularly precise operation for desynchronization of tributary flow.
(6) The systems must be designed to meet the peculiar prevalence of torrential type floods rather than floods of inundation.
(7) As floods are aseasonal, the flood control and warning systems must operate effectively twelve months a year.

Before applying these seven principles to the actual design and construction of the flood control systems in New England, a brief assessment of the type and magnitude of historical flood damage is pertinent.

The Economics of Flood Damage and Flood Control Planning

The constitutional basis of any Federal program, domestic or foreign, contemplates that the broad benefits to the nation shall exceed the cost to the taxpayer. Economic analysis customarily employs statistical methods, and flood damages constitute the raw data from which the economic benefits of planned flood control systems are computed. It must, however, be recognized from the outset that flood damage records are never complete enough to provide an entirely satisfactory basis for computing either a "design flood" or, in consequence, the economic losses which would result in the absence of protective measures. Notwithstanding the startling development of the field of mathematical statistics in the last twenty years, the probable error of computing an accurate design flood from the relatively meager historical records available in even the three hundred years of recorded New England history is unsatisfactorily large. It must be accepted, therefore, that some degree of educated guesswork enters our computations of economic benefits.

Historical records of damaging floods, which extend back to 1635, show that several major floods have occurred in New England in every generation since the days of the early settlements. The most damaging floods have been those of recent years as communities and industrial concentrations have expanded. Five major floods have occurred in New England since the Corps of Engineers began detailed damage surveys in 1927.

Hurricane "Connie" (11-14 August), the first full tropical hurricane of the 1955 season, brought moderate to heavy rainfall of 5 to 9 inches over southern New England. These rains saturated the ground and streams ran bankfull. This storm set the stage for one of New England's greatest flood-producing storms—hurricane "Diane" three days later. From the 17th to the 19th of August, an average of 12 to 14 inches of rain fell in an area of approximately 3500 square miles, extending from the Berkshire Hills in western Massachusetts and Connecticut to the eastern coast. In some areas of central and southern Massachusetts, record depths of 16 to 20 inches of rainfall were recorded. Runoff was heavy over the already saturated ground, and the four major river basins of southern New England—the Housatonic, Connecticut, Thames and Blackstone—produced one of the most devastating floods in the history of New England. More than 90 lives were lost and

damages totalled $530 million. Damages were intensified when small dams washed out in rapid succession. Where bridges were strong enough to withstand the impact of the flood, temporary dams were formed by the pile up of debris: the wreckage of homes, factories, whole lumber yards, railroad boxcars and uprooted gasoline tanks. As the rivers were forced to flow around these debris-dams, destructive new channels were cut through residential and industrial areas. Where bridges succumbed to the force of the flood, destructive debris was carried for miles downstream.

Damages were particularly intense along the tributary streams in this flood. Losses along the main rivers were dwarfed by the damages inflicted by such tributaries as the Naugatuck in the Housatonic Basin, the Quinebaug and French in the Thames Basin, and the Farmington, Westfield and Chicopee in the Connecticut Basin. The Naugatuck River alone, a relatively small stream that flows 40 miles through one of New England's most highly concentrated industrial areas, caused losses estimated at a quarter of a billion dollars, as compared to the half billion dollar damage suffered by New England as a whole. A complex method of economic analysis is employed to balance the benefits against the cost of protection. In largest essentials, the quarter of a billion dollar loss in the valley of the Naugatuck enters in stage, damage and frequency analyses to determine the annual flood loss in the basin. A comprehensive system of flood protective measures is devised which will provide the highest degree of control over the theoretical "standard project flood." Set against the annual benefits computed from flood damage reductions in the basin is the construction cost of the optimum flood control system, amortized over the economic life of the structures, plus interest on the Government's investment, plus the annual costs of operation and maintenance. The ratio of annual economic benefits to annual construction and operation costs computed by this highly oversimplified method constitutes the so-called "benefit-cost ratio" on which the Corps of Engineers base their recommendations to the Congress. A benefit-cost ratio greater than unity is the criterion of a favorable recommendation.

While this method serves admirably for the entire basin flood control system as a whole, it is frequently misunderstood when applied to individual projects in the system. For example, the dikes and flood walls protecting Hartford, Connecticut, were more than justified in the absence of flood control reservoirs in the tributaries of the Connecticut upstream. The very existence of these local protective works around so important a damage center as Hartford, however, automatically reduces the benefit-cost ratio of the uncompleted reservoirs on the tributaries. Whereas the basin system as a whole may have a "B-C ratio" of perhaps two to one, the incremental benefits of a particular structure, in view of the completion of other elements of the system, may well be reduced to near equivalence with the annual carrying costs of its construction. For this reason it is important to consider the individual projects together as component elements of the system of flood control for the basin as a whole. All too frequently and tragically has a worthy project remained "inactive" for years because of an individual "B-C ratio" of perhaps one point two to one (1.2 to 1.0) whereas, considered as an element of the basin system as a whole, it should have carried a "B-C ratio" of perhaps three or four to one. Disastrous floods result most frequently in New England as elsewhere from a false sense of security on the part of the populace. A clear understanding of the method of computing incremental benefits can go far toward eliminating this false sense of security and over-

come the misapprehension that a project bearing a low individual "B-C ratio" is of minor importance. The old adage "a chain is no stronger than its weakest link" applies nowhere more accurately than to New England flood control systems, where numerous elements must be built to operate in unison if disasters of the future are to be prevented.

A storage reservoir of more than 50,000 acre-feet controlling 200 square miles or more is considered a large project in New England. The mean capacity of existing and planned reservoirs is 45,000 acre-feet (Plate No. 2). The largest would impound less than 150,000 acre-feet. In areas of the mid- and far-west, storage capacity of two million acre-feet are not unusual. The Fort Peck Reservoir on the Missouri River, for instance, has a capacity of more than 19.4 million acre-feet. Some reservoirs on the major rivers of America control drainage areas as large as the whole New England region. The largest flood control reservoir yet built in New England, Franklin Falls Reservoir on the Pemigewasset River near Franklin, New Hampshire, impounds only 154,000 acre-feet.

Many of the reservoirs in other parts of the country combine flood control with power and conservation benefits. These multi-purpose projects represent, essentially, a compromise between the dry-bed reservoirs exclusively for flood control and those maintained partially full for power, water supply, recreation, and other purposes. Such reservoirs are practical if they can be built large enough to leave a substantial part of the reservoir empty during periods of predictable flood runoff. Reservoir sites of such size are scarce in New England, and flood runoff is not predictable. Hence multi-purpose benefits in New England rarely apply, and flood control reservoirs must rest economically on their own feet.

Two additional elements of flood control planning should be considered. Watershed treatment or so-called "land management programs" and flood plain zoning which play significant roles in other parts of the country, contribute only minimally to flood control in New England. Soil conservation through contour plowing, reforestation, small detention reservoir construction, and other methods of land management for the control of erosion and watershed runoff are effective in the reduction of small floods. These measures have little effect, however, on major floods. Great floods have occurred as early as 1635, 1683, and 1692, when New England was a virgin wilderness and land management problems were of little consequence.

Flood-plain zoning is, of course, the only positive means of preventing damage. Since all uncontrolled rivers inundate their flood plains periodically, man takes a calculated risk when he builds homes and factories along the banks of streams. The more he builds and encroaches upon the flood plain, the greater the risk of flood damage. Nevertheless, this risk has long been accepted in New England, and under present conditions of development, the cost of flood-plain zoning would involve a major relocation of the industry and population of the region. Recent public awareness of the flood potential of New England streams is evident, however, in the efforts of the state planning commissions and urban development groups to encourage new industry and housing developments to seek higher ground. Again, these efforts though laudatory, can have but minimal affect on flood damage in the foreseeable future.

Principal Flood Control Systems

The Corps of Engineers has formulated comprehensive flood control

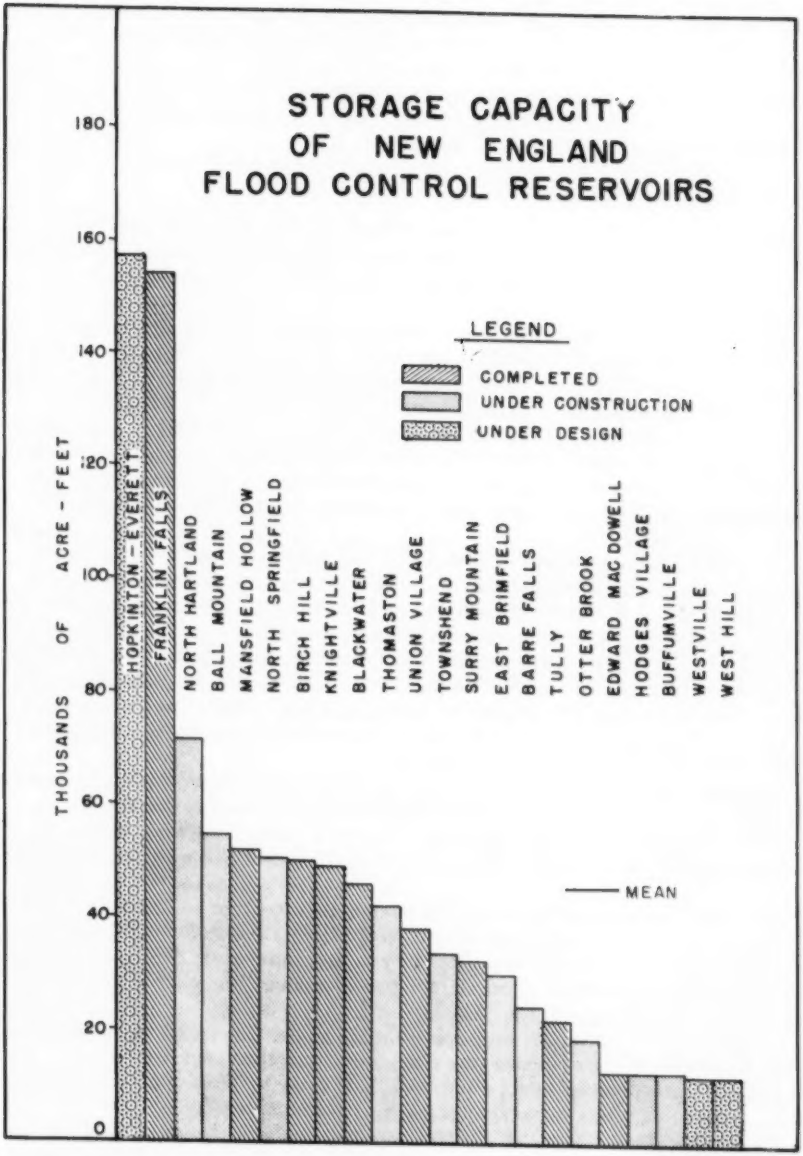

STORAGE CAPACITY
OF NEW ENGLAND
FLOOD CONTROL RESERVOIRS
THOUSANDS OF ACRE - FEET
0
20
40
60
80
100
120
140
160
180
LEGEND
COMPLETED
UNDER CONSTRUCTION
UNDER DESIGN
MEAN
HOPKINTON - EVERETT
FRANKLIN FALLS
NORTH HARTLAND
BALL MOUNTAIN
MANSFIELD HOLLOW
NORTH SPRINGFIELD
BIRCH HILL
KNIGHTVILLE
BLACKWATER
THOMASTON
UNION VILLAGE
TOWNSHEND
SURRY MOUNTAIN
EAST BRIMFIELD
BARRE FALLS
TULLY
OTTER BROOK
EDWARD MACDOWELL
HODGES VILLAGE
BUFFUMVILLE
WESTVILLE
WEST HILL

PLATE NO. 2

systems for each of the five major flood-producing basins in New England: the Housatonic, Connecticut, Thames, Blackstone and Merrimack River Basins. These systems have been authorized by Congress in a number of acts beginning with the Flood Control Act of 1936. Each such system, consisting of a complex of desynchronizing tributary reservoirs and supplemental local protection works, is planned to provide the optimum flood protection that can be economically justified in each basin. The Connecticut and Merrimack River Basins are taken as illustrative.

The Connecticut River Basin

The Connecticut River, the largest in New England, is about 400 miles long and drains approximately 11,200 square miles in four states and Canada. It has numerous minor and sixteen major tributaries, the average draining about 500 square miles.

The Authorized Plan.—The comprehensive system for the Connecticut River Basin consists of 23 reservoirs and 11 local protection works (Table 1). A total of 5 reservoirs and all 11 local protection works have been completed and put into operation at a Federal cost of $34.1 million. The Knightville, Birch Hill, Tully, Surry Mountain and Union Village reservoirs control several of the most damaging of the Connecticut River tributaries and the series of dikes and floodwalls along the river at Northampton, Holyoke, Chicopee, and Springfield, Massachusetts, and at Hartford, Connecticut, shield these local damage centers.

Six reservoirs estimated to cost $47.2 million are in advance design or under construction. Construction of three additional authorized reservoirs for control of the Passumpsic, White and West Rivers in Vermont appears justified under present economic conditions. Economic changes in the basin since the remaining nine reservoirs were planned will require re-evaluation of these projects. As the result of the 1955 floods and consequent authorization of the Northeast Flood Studies, two additional projects were recommended to Congress in 1956, and interim reports are being prepared for other sections of the basin (Plate No. 3).

Development of Basin Plan.—The integrated flood control system presently authorized for the Connecticut Basin was based on hydraulic analyses of experienced floods and damage surveys. Possible local protection works at damage centers were studied and several hundred dam sites were investigated.

The most economical method of flood control appeared to combine tributary reservoirs and local works on the main stem since reservoirs alone would not reduce flood stages sufficiently at the main-stem damage centers, and dikes or floodwalls designed to contain unreduced stages would require impractical heights and great costs.

Analysis of Floods.—The present estimate of the flood-producing potential of the Connecticut River Basin was computed from the flood records of March, 1936, September 1938, June 1947, and January 1949. Analysis of the 1955 floods is part of the current Northeast Flood Studies.

The Connecticut River was divided into successive reaches limited by the mouths of tributaries and hydraulic control points. Both observed and synthetic hydrographs of these reaches with allowances for distance of travel, character of reach, amount of intervening inflow, and relative timing were routed downstream to determine their contributions to main river peaks.

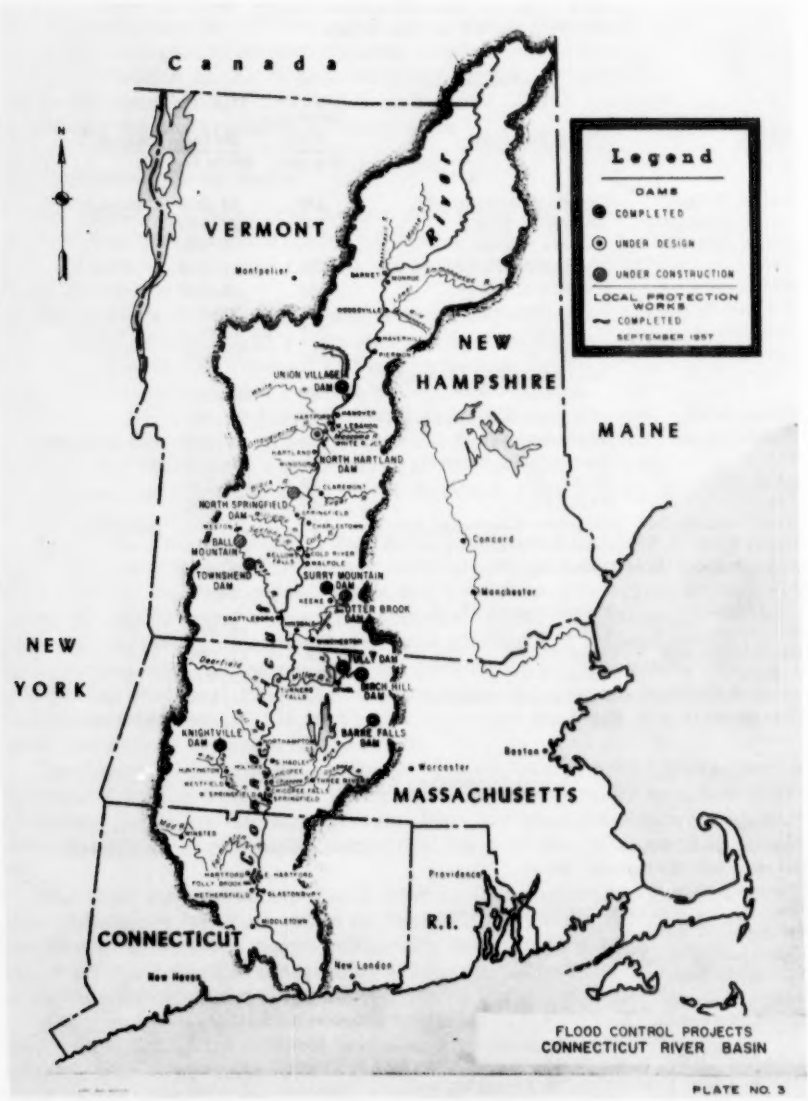
TABLE I
AUTHORIZED FLOOD CONTROL WORKS
CONNECTICUT RIVER BASIN

Reservoirs

Name	River and State	Drainage Area (sq. mi)	Storage (acre ft.)	Status
Union Village	Ompompanoosuc, Vt.	126	38,000	Completed
Surry Mountain	Ashuelot, N. H.	100	32,500	"
Birch Hill	Millers, Mass.	175	49,900	"
Tully	Millers (Tully) Mass.	50	22,000	"
Knightville	Westfield, Mass.	162	49,000	"
Ball Mountain	West, Vt.	172	54,600	Under constr.
Townshend	West, Vt.	278	33,000	"
No. Springfield	Black, Vt.	158	50,600	"
Otter Brook	Ashuelot (Otter Brook), N. H.	47	17,500	"
Barre Falls	Chicopee (Ware), Mass.	57	24,300	"
No. Hartland	Ottauquechee, Vt.	222	71,100	Design in prog.
Victory	Passumpsic (Moose), Vt.	75	106,400	Active
Gaysville	White, Vt.	226	124,150	"
The Island	West, Vt.	40	19,400	"
West Brookfield	Chicopee (Quaboag), Mass.	106	33,000	Inactive
Honey Hill	Ashuelot (So. Br.), N. H.	70	26,200	"
West Canaan	Mascoma, N. H.	80	51,000	"
Alternate to Sugar Hill	Ammonoosuc, N. H.	-	-	"
Brockway	Williams, Vt.	101	37,700	"
Cambridgeport	Saxtons, Vt.	58	21,600	"
Ludlow	Black, Vt.	56	23,900	"
South Tunbridge	White (First Branch), Vt.	102	32,600	"
Claremont	Sugar, N. H.	100	78,300	Deferred

Local Protection Works

Location	River	Status
Northampton, Mass.	Connecticut	Completed
Holyoke and Springdale, Mass.	"	"
Chicopee, Mass.	"	"
West Springfield and Riverdale, Mass.	"	"
Springfield, Mass.	"	"
East Hartford, Conn.	"	"
Hartford, Conn.	"	"
Winsted, Conn.	Farmington (Mad)	"
Hartford, Conn.	Connecticut (Folly Brook)	"
Ashuelot River, N. H. (Snagging and clearing)	Ashuelot	"
Weston Local Protection, Vt. (Snagging and clearing)	West	"


Canada
VERMONT
NEW HAMPSHIRE
MAINE
NEW YORK
MASSACHUSETTS
CONNECTICUT
R.I.
Legend
DAMS
COMPLETED
UNDER DESIGN
UNDER CONSTRUCTION
LOCAL PROTECTION WORKS
COMPLETED
SEPTEMBER 1957
UNION VILLAGE DAM
NORTH HARTLAND DAM
NORTH SPRINGFIELD DAM
BALL MOUNTAIN
TOWNSHEND DAM
SURRY MOUNTAIN DAM
KNIGHTVILLE DAM
BARRE FALLS DAM
Montpelier
Concord
Manchester
Worcester
Boston
Providence
New London
FLOOD CONTROL PROJECTS
CONNECTICUT RIVER BASIN
PLATE NO. 3

This analysis revealed that major floods in the lower basin originated in tributaries of the lower 266 miles of the river. These 14 flood-producing tributaries, in downstream order are:

1. White River, Vt.
2. Ottauquechee River, Vt.
3. Sugar River, N. H.
4. Black River, Vt.
5. Williams River, Vt.
6. Saxtons River, Vt.
7. Cold River, N. H.
8. West River, Vt.
9. Asuelot River, N. H.
10. Millers River, Mass.
11. Deerfield River, Vt. and Mass.
12. Chicopee River, Mass.
13. Westfield River, Mass.
14. Farmington River, Mass. and Conn.

The discharges of the tributaries to the central Connecticut River Basin are closely synchronized and combine to produce the major flood crest in that area (Plate No. 4).

Standard Project Flood.—Studies of great storms in the Connecticut River Basin indicate the probability of a flood somewhat greater than the record flood of March 1936. Protective works must therefore be based on a somewhat greater flood. Depth and distribution of rainfall were studied to determine the "Standard Project Storm." The "Standard Project Flood" was computed from the "Standard Project Storm" by means of mass rainfall curves of the tributary areas, unit hydrographs and flood routings from river channel characteristics of past floods. This Standard Project Flood, combined with the major floods of record, determined the flood control system finally recommended to Congress.

Selection of Plan.—The reservoirs in the presently authorized plan would control 25% of the Connecticut River watershed above the major damage centers on the main stem. An analysis of floods prior to 1955, revealed that reservoir control of the principal flood-producing tributaries in the middle of the basin with runoff from other areas. The flood of August 1955 indicated the need for additional reservoirs in the lower valley tributaries.

The grade of the dikes and floodwalls along the main river is designed for a flood somewhat greater than that of March 1936—after reduction by the authorized system of reservoirs. As long as this upstream reservoir system remains incomplete, these dikes are in danger of failure. Hurricane "Diane" of 1955, for example, would have caused a flood much greater than that of 1936 had it continued up the Connecticut Valley instead of curving out to sea over eastern Massachusetts.

The five existing flood control reservoirs and the six now under construction on the West, Ottauqueehee and Black Rivers in Vermont, the Ashuelot River in New Hampshire and Chicopee River in Massachusetts will reduce both main river flood peaks and tributary damage. The Littleville Reservoir in Massachusetts, and Mad River Reservoir in Connecticut, recommended for authorization as part of the Northeast Flood Studies, will increase control over the southern tributaries. These projects when completed will ameliorate but will not solve the problem of flood control on the Connecticut.

Reservoir Regulation.—The Connecticut River reservoirs are operated as an integrated system during times of flood. Regulation of each reservoir in the system depends upon its location within the basin, its storage capacity, and the origin of the flood. During a major flood, regulation is usually accomplished in three steps. Because flood peaks form quickly in the narrow

TRIBUTARY CONTRIBUTIONS TO FLOOD PEAK ON CONNECTICUT RIVER AT HARTFORD, CONNECTICUT

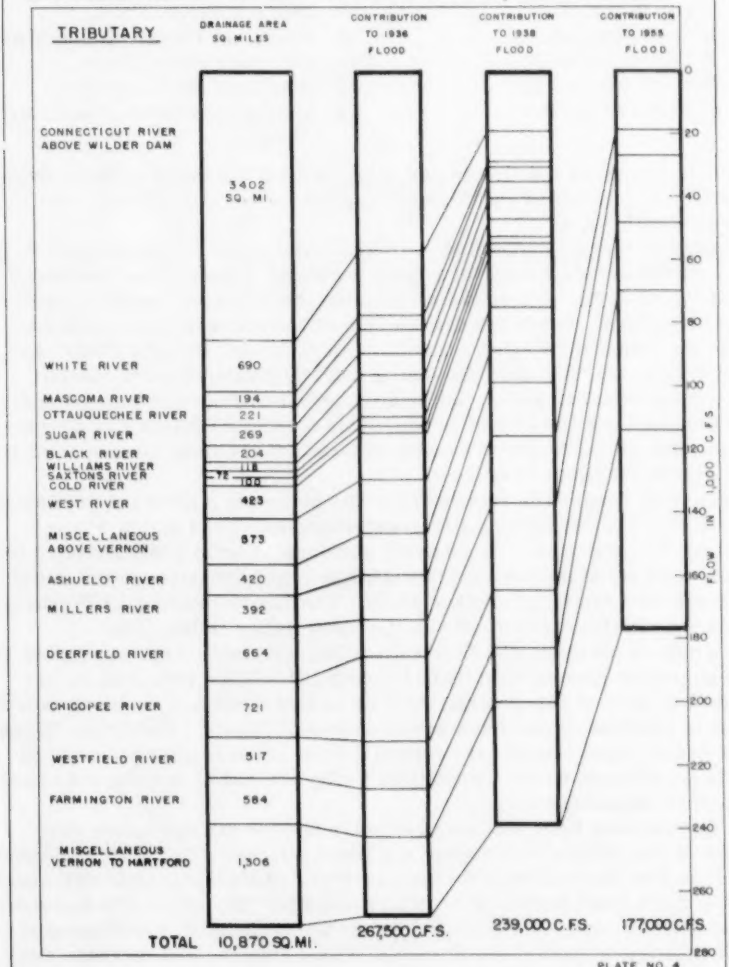


PLATE NO. 4

tributary valleys of the Connecticut Basin, the first step restricts reservoir outflows to protect communities along the tributaries. The second step prevents tributary contributions from augmenting flood peaks on the Connecticut River. In general, the reservoirs remain closed until the flood crest on the main river has passed the tributary mouth. If the reservoirs have sufficient capacity, they remain closed until the flood downstream from the tributary mouth has receded below flood stage. The third step consists of emptying the reservoirs. An integrated system of reservoirs must be emptied in a carefully timed sequence in order to avoid a man-made flood. Following the recession of a flood, each reservoir is emptied as rapidly as the safe channel capacity of the tributary will allow.

A typical example of reservoir regulation in the Connecticut Basin occurred at Knightville Reservoir during the New Year's flood of January, 1949. Knightville Reservoir, completed in 1941, is located on the Westfield River in Massachusetts about 28 miles above its confluence with the Connecticut (Plate No. 5). The reservoir controls a drainage area of 162 square miles. At spillway crest elevation its capacity is 49,000 acre-feet, equivalent to 5.6 inches of runoff. The total capacity of the reservoir was used during this flood.

The flood followed a storm which produced 8 to 10 inches of rainfall over western Connecticut, Massachusetts and southern Vermont. Regulated discharges through the gates began on the morning of December 30, 1948 (Plate No. 6). The gates were closed at 6:00 p.m., when the combined flows from the reservoir, the Middle and the West Branches of the Westfield River, exceeded the safe channel capacity of 5000 c.f.s. downstream at Huntington, Massachusetts. The first step of regulation of Knightville Reservoir reduced flood stages at Westfield, Massachusetts to 32,000 c.f.s. as compared to a peak flow without regulation of 51,600 c.f.s. or a stage reduction of approximately 6 feet at Westfield.

The second step of the operation required the gates to remain closed to avoid contributing to flood peaks on the lower Connecticut River. On January 2, after the discharge at Westfield had diminished below its 10,000 c.f.s. flood stage, and after the flood crest had passed Hartford, Connecticut the gates of the Knightville Dam were opened and the impounded waters gradually released on the recession side of the Connecticut River flows. Releases continued until the reservoir was empty on January 14, 1949.

Benefits.—The 5 existing reservoir projects and 11 local works were built between 1936 and 1952 at a Federal cost of $34.1 million. Cumulative benefits since their construction have amounted to over $85 million. Several of the existing projects have paid for themselves many times over. Knightville Reservoir, for instance, built at a cost of $3,216,500, prevented damages of $6,480,000 during the flood of August 1955 alone.

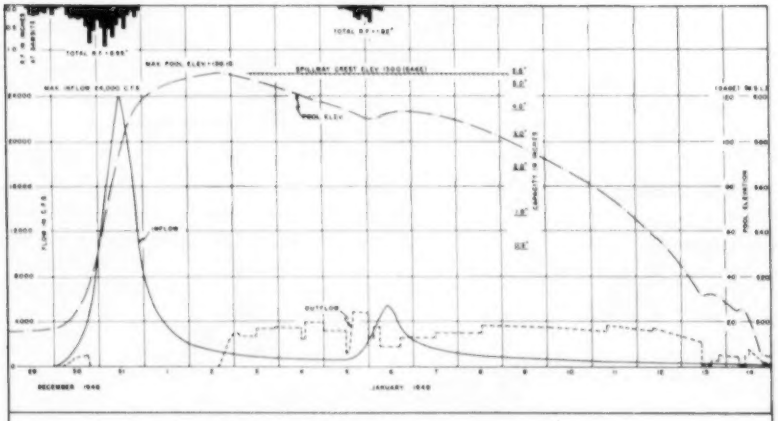
Merrimack River Basin

The Merrimack River Basin extends from the White Mountain region of New Hampshire southward into the central part of Massachusetts (Plate No. 7). The fourth largest river basin lying wholly within New England, it has an area of 5010 square miles, a maximum width of 68 miles and length of 134 miles. Its principal tributaries are the Pemigewasset, Winnipesaukee, Contoocook, Soucook, Piscataquog, Nashua and Concord Rivers.

Flood Damage.—Major floods in the Merrimack are caused by a combination of rainfall and snow melt. Fifty-nine of the 76 largest floods at

N
WATERSHED OF
KNIGHTVILLE RESERVOIR
162 SQUARE MILES
WATERSHED OF
WESTFIELD RIVER
517 SQUARE MILES
SAVOY
PLAINFIELD
ASHFIELD
CUMMINGTON
GOSHEN
WORTHINGTON
CHESTERFIELD
MIDDLEFIELD
BECKET
CHESTER
KNIGHTVILLE
RESERVOIR
HUNTINGTON
MONTGOMERY
BLANDFORD
RUSSELL
WORONOCO
Cobble Mtn.
Res.
WESTFIELD
WEST
SPRINGFIELD
SPRINGFIELD
CONNECTICUT RIVER
GRANVILLE
SOUTHWICK
AGAWAM
Congamond
Lakes
LEGEND
△ U.S.G.S. GAGES
CONNECTICUT RIVER FLOOD CONTROL
LOCATION MAP
KNIGHTVILLE RESERVOIR
WESTFIELD RIVER BASIN
HUNTINGTON, MASSACHUSETTS
SCALE IN MILES
4 0 4

PLATE NO.5



CONNECTICUT RIVER BASIN

KNIGHTVILLE RESERVOIR

OPERATION IN FLOOD OF DECEMBER 1948

WESTFIELD RIVER, HUNTINGTON MASSACHUSETTS

PLATE NO. 6

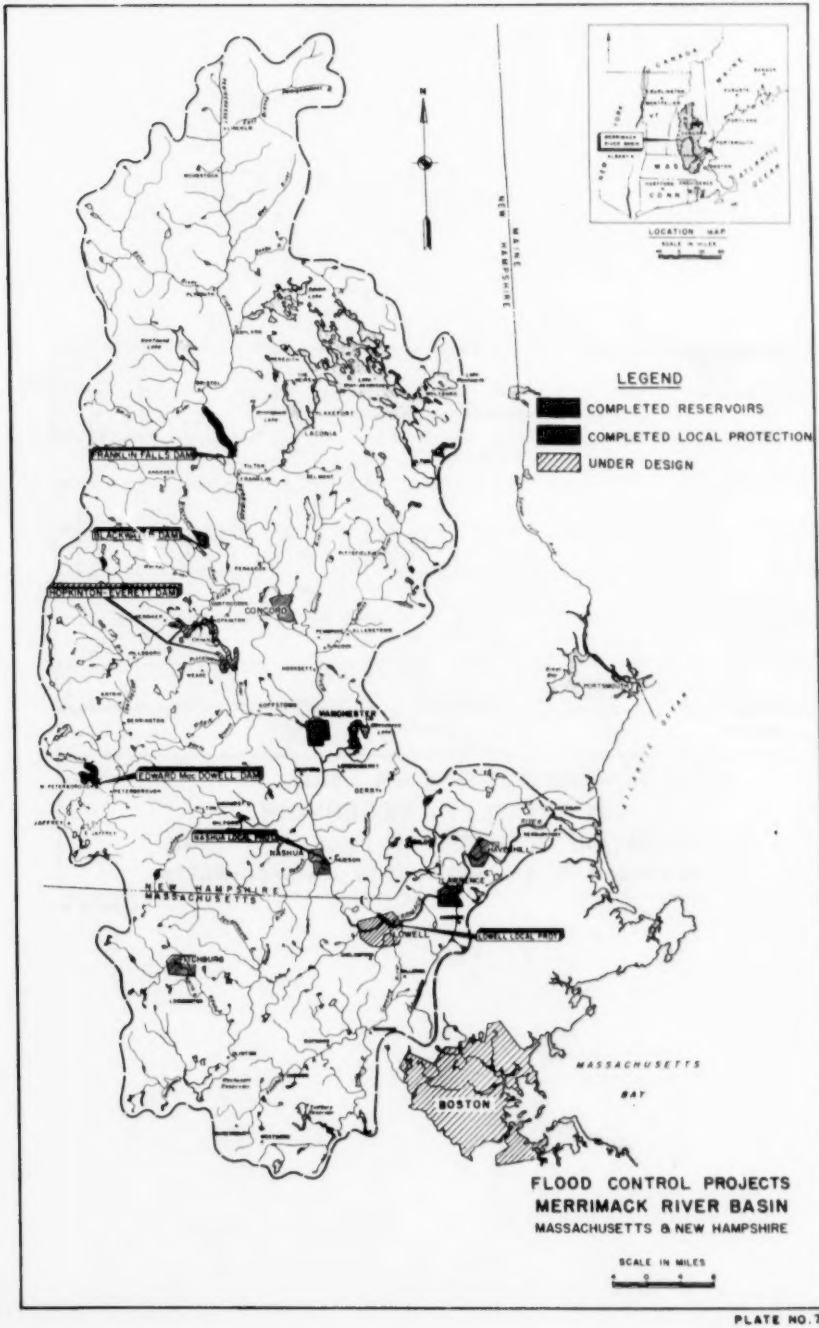

LOCATION MAP
LEGEND
COMPLETED RESERVOIRS
COMPLETED LOCAL PROTECTION
UNDER DESIGN
FRANKLIN FALLS DAM
BLACKWELL DAM
HOPKINTON - EVERETT DAM
EDWARD MacDOWELL DAM
CONCORD
MANCHESTER
NASHUA
LOWELL
LOWELL LOCAL PROTECTION
NEW HAMPSHIRE
MASSACHUSETTS
BOSTON
MASSACHUSETTS BAY
ATLANTIC OCEAN
NEW HAMPSHIRE
FLOOD CONTROL PROJECTS
MERRIMACK RIVER BASIN
MASSACHUSETTS & NEW HAMPSHIRE
SCALE IN MILES

PLATE NO. 7

Lowell, Massachusetts since 1846 were of this nature as was the flood of March 1936, the largest flood on record.

The total losses of $36 million in 1936 were concentrated in the industrial, commercial and residential areas of Haverhill, Lawrence, and Lowell, Massachusetts, and Nashua, Manchester and Concord, New Hampshire. Although all tributaries contributed to the 1936 flood, the heaviest contributions came from the Contoocook and Pemigewasset.

The Authorized Plan.—Following the flood of March 1936, Congress authorized for the Merrimack River Basin a comprehensive flood control system which was modified by the Flood Control Act of 1938. The reservoirs and local works currently authorized are listed below:

AUTHORIZED FLOOD CONTROL WORKS
MERRIMACK RIVER BASIN

Reservoirs

Name	River and State	Drainage area (sq. mi.)	Storage (acre ft.)	Status
Franklin Falls	Pemigewasset, N. H.	1000	154,000	Completed
Mountain Brook	Mountain Brook, N. H.	13	5,300	Inactive
Edward MacDowell	Nubanusit Brook, N. H.	44	12,800	Completed
Blackwater	Blackwater, N. H.	128	46,000	Completed
Hopkinton-Everett	Contoocook and Piscataquog, N. H.	490	157,000	Under Design

Local Protection Works

Location	River	Status
Nashua, N. H.	Merrimack and Nashua	Completed
North Andover and Lawrence, Mass.	Merrimack and Shawsheen	Inactive
Lowell, Mass.	Merrimack	Completed

The five completed works of the authorized plan were built at a Federal cost of $12.8 million. The cost of the Hopkinton-Everett Reservoir, the largest project in New England, is estimated at $34.4 million.

Flood Control Needs.—The completed flood control projects in the Merrimack Basin do not provide sufficient control of the heaviest contributors to flooding on the Merrimack River: the Pemigewasset and Contoocook Rivers. As in the Connecticut Basin, dikes and floodwalls around major damage centers on the Merrimack are in danger of failure under the impact of a major flood. In the Contoocook watershed, Blackwater Reservoir, with a capacity of 6.7 inches of runoff from a drainage area of 128 square miles, effectively throttles the Blackwater River; and the Edward MacDowell Reservoir, which holds 5.5 inches of runoff from 44 square miles of the Nubanusit Brook drainage, protects Peterborough, New Hampshire and reduces flood heights on the Contoocook. The dual Hopkinton-Everett Reservoir project is designed to control both the Contoocook and the Piscataquog.

Contoocook Watershed.--The Contoocook River, the second largest tributary of the Merrimack, draining 766 square miles, rises in Contoocook Lake in southeastern Jaffrey, New Hampshire. The river flows northeasterly for 66 miles to its confluence with the Merrimack River just above Concord, New Hampshire. Containing almost 27 percent of the drainage area upstream from Manchester, its watershed contributes heavily to flood flows on the Merrimack.

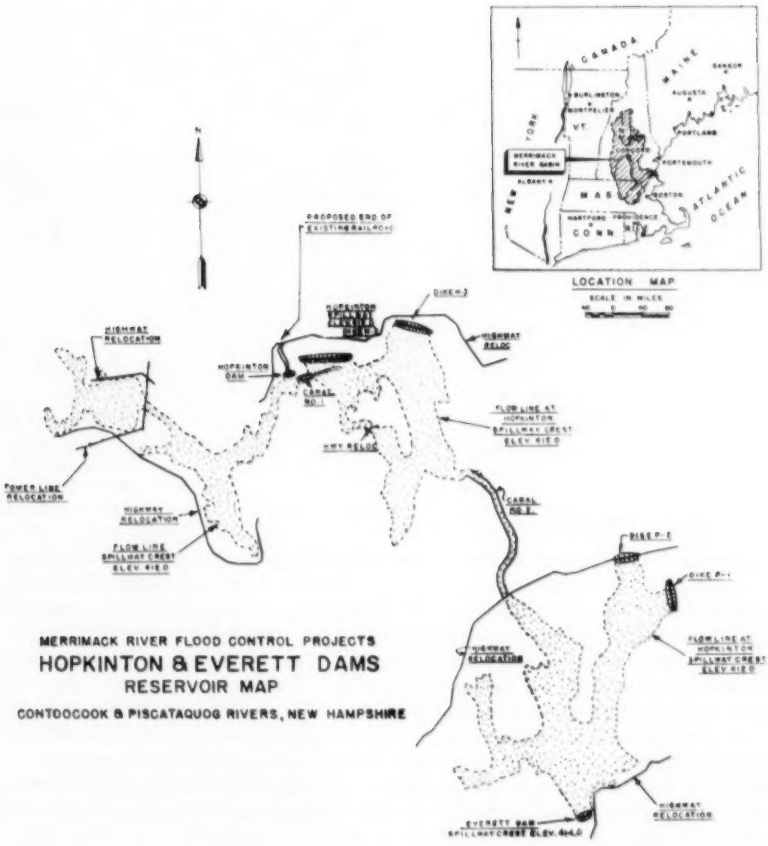
Efforts to find reservoir sites on the Contoocook occupied two decades before 1957. Dozens of sites were investigated. Where reservoir sites were available, drainage areas were too small to provide economic storage. In addition, well-developed transportation systems in the lowland valleys and fertile farming areas made land taking for reservoir sites costly. Some sites which met all other criteria proved disappointing because of foundation conditions. Eventually, a dual project suggested by the New Hampshire Water Resources Board was found economically feasible. Hopkinton-Everett Reservoir was recommended by the Chief of Engineers, and the project was approved by the legislature of New Hampshire with passage of the Merrimack River Flood Control Compact in early 1957.

The Hopkinton and Everett Reservoir will be the largest and most complex flood control project yet undertaken in New England. The plan includes 2 dams, 2 spillways, 4 dikes, and 2 connecting channels (Plate No. 8). Materials required for closure of the reservoirs total over 3.7 million cubic yards, exceeding by 600,000 cubic yards the materials used for Franklin Falls Reservoir, the largest constructed in New England to date. The dual project controls a total drainage area of 490 square miles--426 square miles of the Contoocook watershed and 64 square miles of the Piscataquog. The West Hopkinton, or Contoocook section provides 60,500 acre-feet of storage; the Everett, or Piscataquog section provides 96,500 acre-feet.

By using the adjacent basin of the Piscataquog for flood storage, control of the Contoocook River becomes feasible. The two reservoirs will be joined by a 2-mile connecting channel to permit floodwater of the Contoocook River to flow into the Everett Reservoir on the Piscataquog, where 61% of the total storage of 157,000 acre-feet is provided.

Reductions and Benefits.-- A recurrence of the record flood of 1936 would cause losses estimated at $153 million in the Merrimack River Basin. The flood of August 1955 was, fortunately, a "near miss." Had hurricane "Diane" continued about 100 miles further north before it turned seaward, the resulting flood would have caused damages far in excess of this record figure.

The five completed flood control projects would reduce the losses of a flood comparable to that of 1936 to $81 million. A flood of the 1936 level would, nevertheless, endanger areas behind the dikes and floodwalls of the Merrimack River. At Lowell, Massachusetts a flood of this magnitude would be successfully contained by the local works; but at Nashua, New Hampshire the Merrimack would encroach one foot on the design freeboard, and at Haverhill, Massachusetts the flood would overtop the wall if flashboards were not used. The Hopkinton-Everett Reservoir, in combination with the existing reservoir units of the flood control system, would reduce the recurring 1936 flood levels well below the design grade of the dikes and floodwalls. Reductions of flood stages, ranging from as much as 15.5 feet at Manchester, New Hampshire to 9 feet at Lowell and Lawrence, would save nearly $40 million of damage.



MERRIMACK RIVER FLOOD CONTROL PROJECTS

HOPKINTON & EVERETT DAMS

RESERVOIR MAP

CONTOOCOOK & PISCATAQUOG RIVERS, NEW HAMPSHIRE

PLATE NO. 8

Other Basins

The comprehensive flood control systems of the Connecticut and Merrimack River Basins are typical of those in the other river basins of New England. The Housatonic, Thames, and Blackstone River Basins are under review and several units of the authorized systems are currently under construction.

In the Naugatuck watershed of the Housatonic Basin there are three projects under design or construction, representing a total Federal investment of $18.3 million. The largest unit, Thomaston Reservoir, now under construction at an estimated cost of $17.5 million, would have prevented $77.3 million of loss sustained in the Naugatuck Valley during the flood of August 1955.

The authorized plan for the Thames River Basin includes seven reservoirs and one local work, representing a potential Federal cost of $46.8 million. One reservoir is completed and four reservoirs and the local project are under construction or design. In the nearby Blackstone River Basin, three projects are under design and construction at an estimated cost of $13.6 million. These projects include one reservoir, a river diversion project in Worcester, Massachusetts and local works in Woonsocket, Rhode Island.

In summary, there are now 62 authorized flood control projects under the jurisdiction of the New England Division of the Corps of Engineers. The latest estimate of the cost of the authorized projects is $312.5 million. Twenty-six units have been completed at a Federal cost of $56.6 million, and eighteen others, estimated to cost $137 million, are under design and construction.

Flood Control Compacts

The states located within the Connecticut and Merrimack River Basins have agreed in inter-state compacts to cooperate in solving their flood control problems. The Connecticut River Flood Control Compact was approved by the United States Congress in 1953, and the Merrimack River Compact was approved in 1957. Without these compacts construction of the comprehensive reservoir systems in these basins would have been impossible.

Few towns in which flood control reservoirs were located received flood control benefits to compensate for tax and economic losses. Under these compacts the states have agreed to the construction of reservoirs in the Connecticut and Merrimack River Basins, the states benefiting from flood protection agreeing to pay a portion of the tax loss, economic loss and damage to lands and property involved.

PART II

Hurricane Tidal Flooding

In contrast to the orthodox river flood control problem in New England, the protection of coastal areas from hurricane induced tidal inundation constitutes a revolutionary development in flood control in the United States. Aside from the Galveston flood wall, no such projects have yet been undertaken in this country. As a result of the Hurricane Survey authorized by

Congress in 1955, two such pioneering projects in New England have been recommended to Congress this year.

The tidal floods induced by hurricane "Carol" in August 1954 caused damages of approximately $300 million and a loss of 60 lives in the coastal areas of the northeast. In June 1955, Congress authorized the Chief of Engineers, in cooperation with the Department of Commerce, the Weather Bureau, and other Federal agencies to conduct a survey of the eastern and southern seaboard of the Atlantic and Gulf coasts. The survey was directed to include compilation of data on the behavior and frequency of hurricanes, determination of methods of forecasting and improving warning services, means of preventing loss of life and damage to property, and examination of the economic justification of proposed breakwaters, seawalls, dikes, dams and other structures, warning services or other measures which might be required.

Hurricane surveys have been completed for Narragansett Bay and New Bedford Harbor. Studies of the Connecticut and southern Rhode Island coasts are now in progress. The New England Division has relied heavily on other Corps of Engineer units: the Beach Erosion Board, Washington, D. C., aided by the Texas A & M Research Foundation, and the Waterways Experiment Station at Vicksburg, Mississippi. A number of private engineering firms as well as the Weather Bureau, Department of Commerce; the Fish and Wildlife Service, Department of the Interior; the Public Health Service, Department of Health, Education and Welfare; and the Coast and Geodetic Survey have assisted in these studies. Many state agencies, particularly the Narragansett Marine Laboratory of the University of Rhode Island, have contributed.

Hurricanes and Hurricane-Flood Damages.—Hurricanes, the most destructive of all storms, occur most frequently in the months of August, September and October. These gigantic cyclonic storms form over tropical waters, move slowly in a northwesterly direction, and travel northward at increasing velocities. As they strike the coastline, they cause great damage from floodwaters and storm waves.

Early accounts of hurricanes in the area are brief, but since 1815, and particularly in the last 50 years, accounts have become increasingly complete. The contemporary period from 1901 to the present is marked by fairly complete and scientific accounts of hurricanes. The earliest hurricane of record in New England occurred on August 15, 1635. Since this date there is a record of 63 hurricane occurrences, 24 between 1635 and 1900, and 39 since that time. The apparent concentration of hurricanes in this century, however, is attributed to the lack of records on early storms and is not believed indicative of a trend toward greater frequency.

The most damaging hurricane floods recorded were those of August 1635, August 1638, August 1788, September 1815, September 1869, September 1938 and August 1954 (Carol). The floods of 1635 and 1638 were probably the greatest in the history of New England. The storm flood of 1815, which carried shipping onto the streets of Providence, Rhode Island was probably less damaging than the hurricane floods of 1938 and 1954.

The three most damaging hurricanes since 1900 occurred in the 17-year period between 1938 and 1954. The 1938 storm took 500 lives in New England and inflicted damages along the coast, mainly from tidal flooding, of nearly $300 million. The 1954 storm took 60 lives with damages of the same magnitude as the 1938 storm. Both storms were nearly coincident with high tides, and produced flood levels along the coast as much as 15 to 16 feet above mean sea level. The hurricane of September 1944, although a severe storm,

struck at low tide and flooding was, therefore, less severe.

Protection of Exposed Coasts

The hurricane survey in New England is concerned with two types of coastline: the sheltered harbors, estuaries and bays like New Bedford Harbor and Narragansett Bay and the exposed coastline, with long stretches of barrier beaches, broken by occasional tidal inlets. The southern Rhode Island coast is a typical example of the latter. This coast, directly exposed to the open ocean, receives the full impact of hurricane waves generated over a fetch of many miles. Protection of these exposed areas differs from that of sheltered harbors and bays, where the construction of barriers, dikes and sea walls is feasible. Hurricane protection along extensive beaches is closely related to beach erosion control and the stabilization of inlets leading to the interior lagoons and salt ponds. Exposed sections of the Rhode Island coast, including Misquamicut, Charlestown Beach and Point Judith are problem areas now under study.

Protective Measures.—There are four basic methods of protecting exposed coasts against hurricane flooding: (1) raising of the natural barrier beach above flood level or construction of dikes of earth and stone: (2) construction of concrete seawalls, (3) building stone revetments to protect headlands and exposed slopes, and (4) the zoning of shore areas subject to hurricane tidal damage. In some areas the growth of natural dunes has been aided by the use of snow fences which retard the sand drift, and vegetation has been used to stabilize the slopes. However, it is doubtful if such methods, in New England, could build up the height and volume of embankment necessary to provide protection against a 12-foot hurricane surge surmounted by 10 to 20-foot storm waves. Some form of rock or pavement is required to armor exposed slopes where extremely flat slopes are impractical.

The regulation of tidal inlets to prevent flooding of the salt ponds and marshes behind barrier beaches is another aspect of the protection of exposed coasts. Storm waters have flowed through the inlets leading into Point Judith Pond and Quonochontaug Pond on the southern Rhode Island coast, but the principal flooding has occurred from overtopping and breaching of the barrier beaches. Once the barrier beach has been restored or raised to prevent overtopping, a stabilized inlet will serve to restrict the flow of water into the salt ponds. Narrowing of the inlets to reduce flooding may prove practical, unless such narrowing creates currents too swift to permit normal small boat navigation. Navigation and sluice gates may be considered if ungated openings are inadequate.

Hurricane Warnings and Emergency Measures.—It is clearly impossible to protect the entire New England coast against hurricane tidal floods. Flood losses in unprotected areas can be reduced through more effective forecasting and warning systems and flood mobilization programs.

Tidal-Flood Protection for the Narragansett Bay Area

The Problem.—Hurricane "Carol" of August 31, 1954 caused losses of approximately $90 million in Narragansett Bay (Plate No. 9). Nineteen lives were lost in the bay and southern coastal areas of Rhode Island. About half of the damage occurred in the upper bay north of Conimicut Point, and damages in the Mt. Hope Bay area of Massachusetts amounted to $7 million.

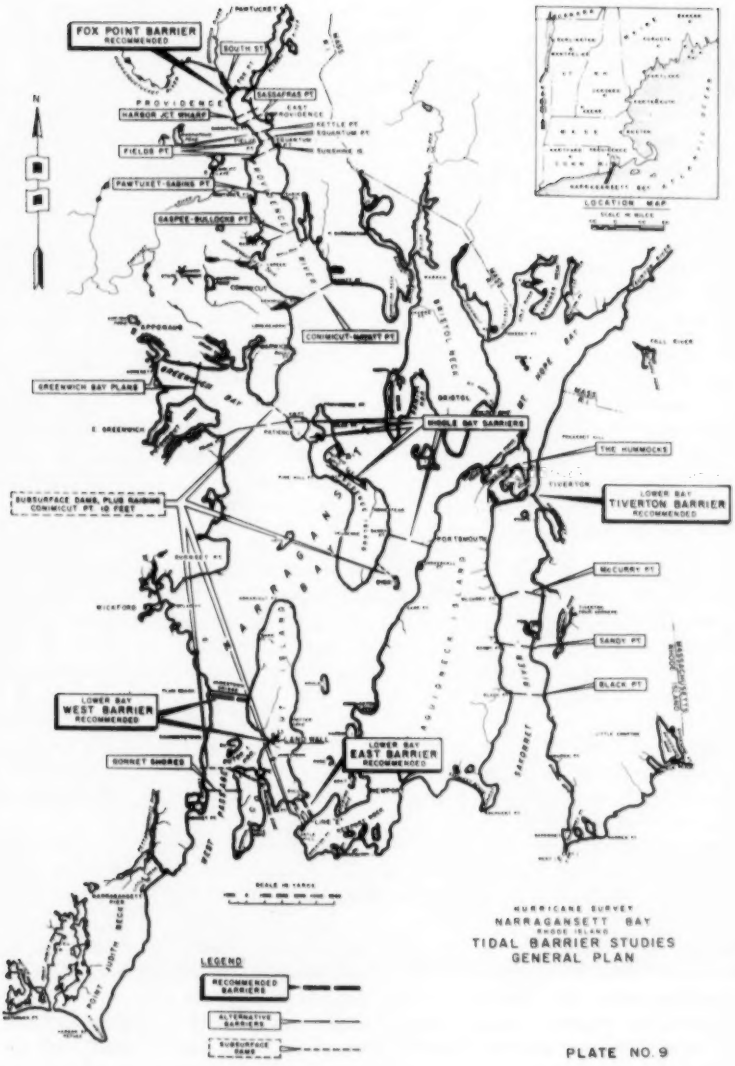

FOX POINT BARRIER
RECOMMENDED
PROVIDENCE
HARBOR JCT WHARF
FIELDS PT
PAWTUXET-SABINS PT
GASPEE-BULLOCKS PT
CONIMICUT-GOULD PT
GREENWICH BAY PLANS
SUBSURFACE DAMS, PLUG GASPEE CONIMICUT PT. TO FEET
LOWER BAY
WEST BARRIER
RECOMMENDED
BONNET SHORES
LOWER BAY
EAST BARRIER
RECOMMENDED
NEWPORT
MIDDLE BAY BARRIERS
THE HUMMOCKS
LOWER BAY
TIVERTON BARRIER
RECOMMENDED
McCURRY PT
SANDY PT
BLACK PT
LOCATION MAP
LEGEND
RECOMMENDED BARRIERS
ALTERNATIVE BARRIERS
SUBSURFACE DAMS
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
TIDAL BARRIER STUDIES
GENERAL PLAN

PLATE NO. 9

Approximately one-third of the total damage was sustained in the business center of the city of Providence, which was flooded with 6 to 8 feet of tidal water.

The 1938 hurricane flood was more severe, causing levels about one foot higher than the 1954 storm. Depths of flooding ranged from about 11 feet msl at Newport to 16 feet msl at Providence. Two hundred and sixty two lives were lost in the state of Rhode Island alone. A storm of this magnitude recurring in 1957 would cause losses estimated at $120 million.

Recommended Plan.—For the protection of Providence and Narragansett Bay, at least 15 plans for hurricane barriers, involving 25 locations, were proposed soon after the 1954 hurricane. A hydraulic model of Narragansett Bay, 250 feet in length and 100 feet in width, built at the Waterways Experiment Station in Vicksburg, Mississippi, was a valuable tool in studies of tidal conditions in the bay and the effectiveness of barrier reductions of hurricane flood levels (Plate No. 10). After completion of the engineering studies, including foundation exploration and model tests of the most practical plans, a two-unit solution of the problem was recommended in the report by the Division Engineer:

(1) Construction of a concrete barrier and pumping station across the Providence River at Fox Point for the protection of the business center of Providence.
(2) Construction of rock-fill barriers across the East and West Passages of Lower Narragansett Bay, and a barrier across the Sakonnet River at Tiverton (subject to more detailed design studies of structures and effects on navigation, pollution and fisheries) for the general protection of the entire Narragansett Bay area.

Fox Point Barrier.—A concrete gravity dam about 1100 feet long to be constructed across the Providence River, the mouth of the Blackstone, from Henderson Street to Fox Point would contain sluice gates to pass the normal river flow. A pumping station was designed for passing river flow into the bay under flood conditions when the gates were closed. Reinforced concrete walls at either end would tie into high ground.

The 1938 hurricane flood level at Providence was measured at 15.7 feet above msl. The design flood would be 3 feet higher. Design of the barrier with top elevation at 22.5 feet above mean sea level allows for 3.8 feet of freeboard. A 37-foot wide deck has been set at elevation 12.5 above msl for wharfage or highway use.

Sluice gates would prevent entry of floodwaters from the bay. These gates would pass normal tides, and would also be capable of discharging all river flow, including full flood runoff of the Blackstone River Basin, during normal tides.

The concrete barrier dam was designed on piles driven to approximately 60 feet below msl. Borings show that the foundation is of variable silts, sands, and gravels overlaid by a heavy deposit of organic silt.

The pumping station would contain 5 large pumps with a combined capacity of 8000 cubic feet per second under a head of 22 feet. Under conditions of a simultaneous hurricane surge and a river flood, the pumps could draw the pool down to a safe elevation.

Lower Bay Barriers.—The East Barrier was designed as a massive stone structure 3200 feet long extending across the East Passage at Newport. It would be constructed with a quarry-run stone core, capped and faced with

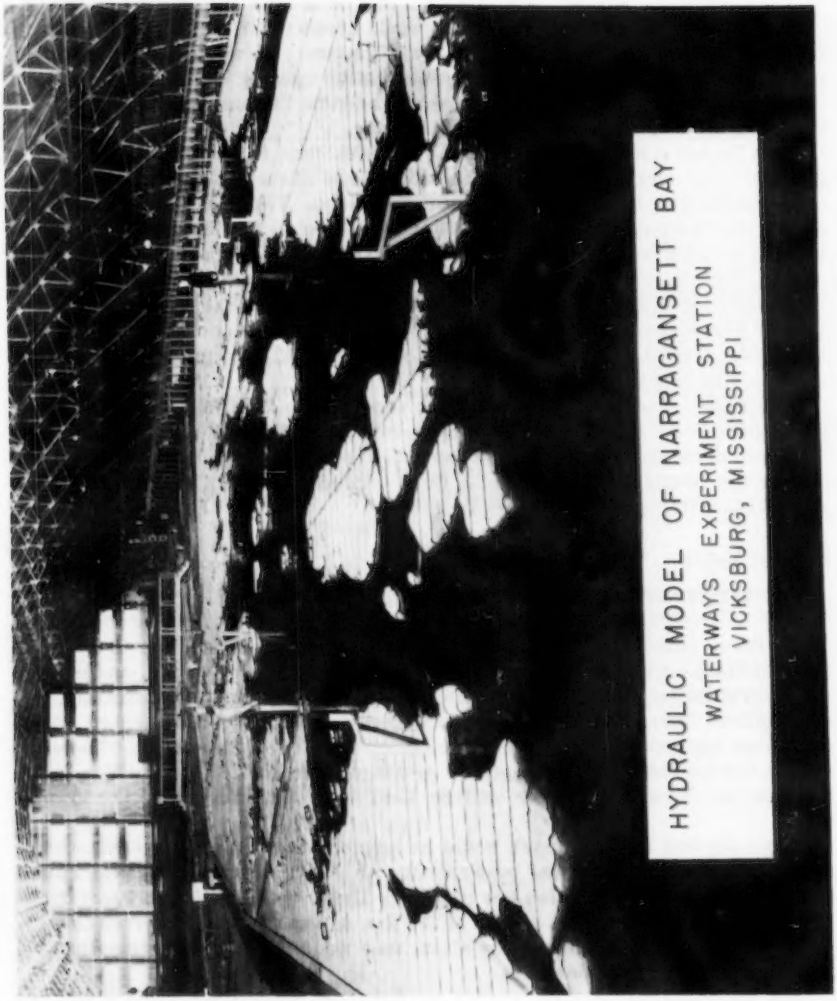

HYDRAULIC MODEL OF NARRAGANSETT BAY
WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

PLATE NO. 10

heavy derrick stones. The top would be 20 feet wide at an elevation 22 feet above msl with side slopes of 1 vertical on 2 horizontal. The maximum base width of the structure would be more than 700 feet in a water depth of 165 feet. It would be oriented at right angles to the existing navigation channel, with a navigation opening 1000 feet wide and 50 feet deep at mean low water.

The West Barrier, 7100 feet long, would cross the West Passage about 600 feet south of the Jamestown Bridge. Construction would be of the same general type as the East Barrier. A navigation opening 400 feet wide and 40 feet deep at mean low water would be centered on the existing ship channel and navigation span of the Jamestown Bridge.

The Tiverton Barrier at the head of the Sakonnet River would consist of an earth-and-rock fill dike along the shorelines of the Island Park and Tiverton areas and a crossing at the Old Stone Bridge. Total length would be about 9500 feet. A navigation opening 100 feet wide and 30 feet deep below mean low water would be closed during hurricane periods by a pair of sector gates.

Cost and Effectiveness.—The Fox Point Barrier would provide almost complete protection against hurricane tidal flooding for the major part of the City of Providence now vulnerable to flooding. Annual benefits of the Fox Point plan are $1,733,000; with annual costs of $732,000, the benefit-cost ratio is 2.37 to 1.0.

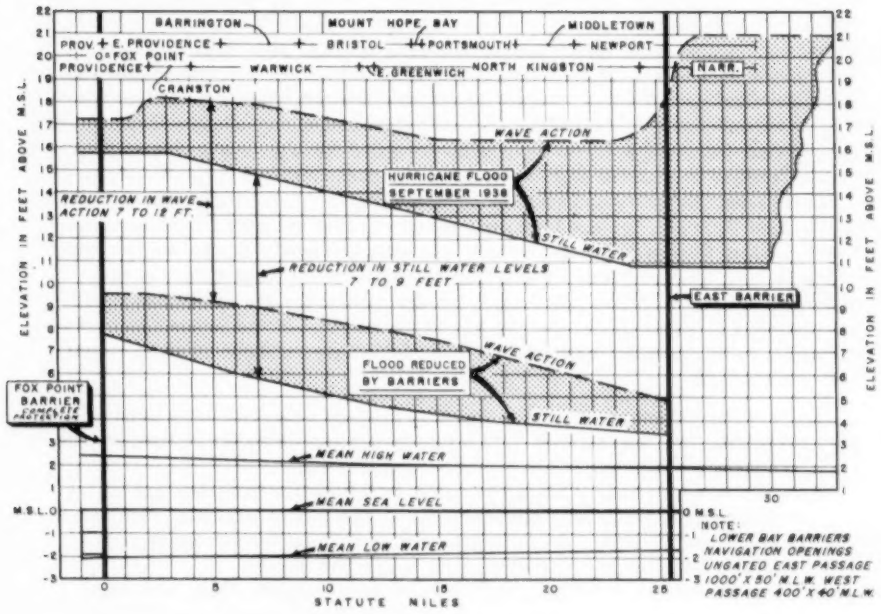
The Lower Bay barriers would have reduced the 1938 flood surge at the head of the bay from 15.7 to 7.7 feet above msl. (Plate No. 11). At Newport, the level would have been reduced from 11.0 to only 3.6 feet above msl. The cost of the Lower Bay Barrier plan is estimated at roughly $70 million. Annual benefits are estimated at $4,267,000.

Barrier Gates

Any barrier against salt-water flooding at the mouth of a river must, of course, be designed to pass the river flow into the sea. A flood of the Providence River coinciding with a hurricane tidal surge from the bays creates a dual problem. Even normal river flow, tidal flushing, navigation, and marine life migration require openings in the barriers which can be closed if necessary against hurricanes and the river waters pumped over the barrier. An important aspect of the hurricane protection problem is the design of openings in the barriers. Three types of openings were studied for the Fox Point Barrier, the Tiverton Barrier, and the East and West Barriers of the lower bay.

Since the Fox Point Barrier would be located near the head of navigation, no provisions for shipping or boat passage above the barrier are contemplated. Four simple sluice gates are planned. These would be 20 x 24 foot drop-type gravity-closure gates, hinged at the top, normally open for river flow and tidal flushing. At a hurricane alert, they would be closed by tripping latches to release the free ends.

For the Tiverton Barrier, across the Sakonnet River from Portsmouth to Tiverton, to provide for tidal interchange and the passage of small boats into Mount Hope Bay, an opening 100 feet wide by 15 feet deep would be closed by a pair of sector gates. These gates are sectors of a cylinder pivoted vertically at their axes with the curved skin plates sealing the opening. This type of gate, used in many installations of the Corps of Engineers, was chosen because of its proven dependability in this range of opening sizes and operation under conditions of possible reversal head.



HURRICANE SURVEY

NARRAGANSETT BAY

RHODE ISLAND

HURRICANE FLOOD LEVELS
SEPTEMBER 1938 FLOOD REDUCED BY
FOX POINT AND LOWER BAY BARRIERS

PLATE NO. 11

Other types of gates considered for the Tiverton Barrier were miter gates, hoist types, and experimental float-up designs. The sector gate could be used directly, however, without changes in design, model studies, or development of new types of operating machinery.

The East and West Barriers of the lower bay constitute the most dramatic engineering challenge of any of the proposed hurricane structures. Plans provide for navigation openings of 1000 feet in the East Barrier and 400 feet in the West Barrier. Tests on the hydraulic scale model of Narragansett Bay at the Waterways Experiment Station at Vicksburg, Mississippi, and analytical studies have indicated that barriers with opening of this size would have reduced the 1938 hurricane tide level by about 8 feet within the bay. The openings, in addition to meeting the needs of navigation, would also permit tidal flushing for other needs. A greater flushing rate of pollutants to obtain water quality and fish and shellfish protection may require sluicegates in the West Barrier, such as have been designed for Fox Point. Such gates would present no unusual engineering difficulties.

Larger navigation openings have been suggested by the Navy, particularly in the East Barrier, for the free passage of various elements of the Atlantic Fleet. If this becomes a requirement, the only presently known way in which it can be met without lowering the degree of hurricane protection, is by means of gated navigation openings. Design, construction, operation, and maintenance of gates of the size required for control of the lower bay barrier openings would be without precedent. The following methods of closure have been considered:

(1) Floating Caissons. Various types of floating caissons have been studied. In general, they would be too big and unwieldy to handle, and unstable in the high winds of hurricane storms.

(2) Roll-out Caissons. This type of gate has been investigated for some installations, including the new locks at the Panama Canal. For use in lower bay barriers, where they would be subject to hurricane conditions, they would be prohibitively large and costly.

(3) Float-up Gates. Of various models of float-up gates that have been studied, the most promising is a type of bottom-hinged cantilever. This gate has a vertical transverse section resembling the conventional gravity type of retaining wall. The interior is cellular and watertight with vertical girders on the main structural members. The gate is hinged to the anchorage on the ocean or storm side to permit rotation of the gate and to develop tension. The upstream side of these vertical girders transmits downward pressure to a bearing surface, and the vertically placed girders act as cantilever beams against the anchorage.

The gate rests below the navigational channel when not in use, and it is raised to its upright position by flotation. By reverse sequence, it is lowered to its horizontal position. The process of raising the gate consists of emptying the cells of the structure by blowing out the water with compressed air, allowing the gate to surface approximately upright, and then shifting the center of buoyance of the transverse section to cause further rotation and seating. This is accomplished by filling or emptying the ballast tanks in proper sequence. Many of the details are still to be worked out, but the size, design and operating conditions of any gate planned for the lower bay barriers appear to be the chief difficulties.

Some of the design problems still awaiting solution are those of control of impact forces when the gate is seated in erect position, control of the gate under wave or tidal forces at the point of extreme buoyancy, and dampening the shock when the gate comes to rest on the bottom. Other problems involve hinge and foundation designs, and mechanical systems for operating the gate with compressed air.

Investigations have been made of the use of hydraulic jacks and also of mechanical follower systems for exerting more positive control over the movement of the gate at critical phases of its operation, and these methods have shown some promise.

The subject of gate design for hurricane tidal barriers is, in short, a complex problem with a variety of solutions. The design of such gates under differing combinations of natural forces, foundation conditions, navigational requirements, construction and operational difficulties remains one of the more important aspects of protection against hurricane tidal flooding, as well as one of the most challenging problems for professional engineering research.

Paper 1518 January, 1958 WW 1

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

ST. LAWRENCE SEAWAY, 27-FT. CANALS AND CHANNELS[a]

W. Grothaus[1] and D. M. Ripley[2]

ABSTRACT

The 27-foot St. Lawrence Seaway is described. A brief historical summary is followed by a discussion of the basic criteria adopted in the design of the Seaway. The dams, locks, and channels of the entire project are then described in terms of how they meet these hydraulic criteria.

SYNOPSIS

The St. Lawrence River in its upper and lower reaches from Ogdensburg, New York and Prescott, Ontario to Lake Ontario, and from Montreal to the sea, respectively, has been comparatively easy to navigate and improvements for navigation have not been retarded by complex schemes nor exorbitant expenditures. Rather, these reaches have been developed progressively over the years in step with the economics of the shipping for which they are particularly adopted. Consequently, the work involved in bringing these reaches into phase with the present project is not difficult and is required only from Ogdensburg to Lake Ontario, and at the Welland Ship Canal.

The intermediate sections of the river wherein lie the rapids and power sites have long needed development for navigation, but the complex multipurpose plans for power and navigation, and resultant expensive works, have caused a lag behind the need. The requirements for additional power development by both countries, Canada and the United States, provided the stimulus which has brought about the start of the present project.

The basic criteria which have guided the design of the 27 foot channels and canals are not a recent innovation. The planning reports prepared and issued

Note: Discussion open until June 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1518 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 1, January, 1958.

a. Paper presented at a meeting of the American Society of Civil Engineers, Buffalo, N. Y., June, 1957.

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2. Hydraulic Engr., St. Lawrence Seaway Authority, Montreal, Canada.

over the last 30 or 40 years contain many of the criteria which are being followed today in the improvements to the river from Montreal to Lake Ontario and to the Welland Canal.

The criteria have been modified to meet certain conditions in both the multipurpose and navigation-only reaches. However, the modifications have not lessened the initial requirements, but in most cases have raised the standards, resulting in a more improved Seaway.

This paper contains the basic criteria and their source, and describes how they were applied in the design of the navigation channels and canals in the various reaches of the river and Seaway to meet the conditions imposed by nature and man-made improvements.

Historical Summary

Until the start of the present St. Lawrence Seaway project, the development of the St. Lawrence River for navigation has been principally a Canadian enterprise. Consistent with traffic demands, the St. Lawrence River from the Atlantic Ocean to the Port of Montreal has been improved so that there is now available a channel 35 feet in depth and at least 550 feet in width. Improvement of this section of the river has consisted chiefly of enlargement of the natural river channel, and the characteristics of the river have permitted a progressive improvement not far out of step with the demand of traffic. However, between Lake Ontario and Montreal, the characteristics of the river do not lend themselves to a gradual improvement, and the development of the river in this section for navigation in the last one hundred years has been a series of reconstruction projects, each resulting in facilities larger and more efficient than the previous ones.

Navigation in the St. Lawrence between Prescott, Ontario and Montreal, a distance of 119 miles, is presently limited to the capacity of the 14-foot draft canal system, which consists of a series of twenty-one locks, each 270 feet in length and 45 feet in width, with canals having a bottom width of 80 feet. It is in this section of the river that the major part of the Seaway construction is located. Although there has been a need for a further development of the St. Lawrence River for navigation for some time, and such development has been inevitable, the beginning has been hastened by the development of the river for power purposes—first at the Soulanges Section, where a project was begun in 1929, and now at the International Rapids Section. These power developments minimize the number of locks required and the length of confining canal reaches, thus reducing the cost of the navigation project and increasing the efficiency of operations.

The St. Lawrence Seaway project is essentially the standardization of the navigation facilities between Lake Erie and the Port of Montreal, and, in giving consideration to the manner in which it has evolved, it may be divided into three major areas of activity, namely, the Welland Ship Canal, the International Section of the St. Lawrence River, from Lake Ontario to St. Regis, New York, and the all-Canadian portion from St. Regis, New York to Montreal, Quebec. (See Fig. 1)

Welland Ship Canal

The Welland Ship Canal was constructed in its present form by the Government of Canada during the period 1913 to 1932. It overcomes with a series of eight locks and connecting canals the 326 foot differential between Lake Erie and Lake Ontario. The present canal is the modern version of a canal system which had existed at approximately the same site for well over one hundred years, and it requires only the deepening to Seaway draft and minor widening to bring the system up to Seaway standards.

International Section

The design of the Seaway features which are now under construction in the International Section of the St. Lawrence River has evolved directly from the plans submitted in January 1941 by a Board of Engineers consisting of representatives of the United States and Canada. This Board, which was appointed by the governments of the two countries to scrutinize and investigate the various plans submitted over the years for the development of the International Section of the St. Lawrence River, described in their report and recommended that a plan known as the "238-242 Controlled Single Stage Project" be adopted. It was the opinion of this Board that the plan was "the best from an engineering and economical point of view, bearing in mind the requirements of navigation and power and the protection of downriver interests."

The Board of Engineers' report was the nucleus of the Great Lakes - St. Lawrence Basin Agreement of March 1941. This agreement, which failed of ratification, contained provision for the construction of an over-all project from the head of navigation in the Great Lakes to tidewater below Montreal. The agreement provided, among other things, for equal division of power in the International Section, the sole right of Canada to power in the all-Canadian section of the St. Lawrence River, the deepening of the Welland Ship Canal, and improvements in the all-Canadian section for 27 foot navigation by Canada and improvements in the International Section in accordance with the 238-242 Controlled Single Stage Project plan.

Although never adopted as a comprehensive plan, the essential principles of the 1941 agreement are being carried out as a result of legislative action taken by each country with respect to the Seaway project and by the Order of Approval of the power project granted by the International Joint Commission.

Public Law 358, 83rd Congress, Second Session, approved May 13, 1954, created the Saint Lawrence Seaway Development Corporation and authorized and directed the Corporation to construct, operate and maintain in the United States territory deep-water navigation works necessary to bypass the powerhouse and dams to be constructed by the Power entities and to perform the necessary dredging in the Thousand Islands Section. In July 1954, the St. Lawrence Seaway Authority Act, enacted previously by the Canadian Parliament, was proclaimed in force. This Act created the St. Lawrence Seaway Authority and gave authorization to construct, operate and maintain deep-water navigation works between Lake Erie and the Port of Montreal, either wholly in Canada or in conjunction with the works undertaken by an appropriate authority in the United States. In both acts, deep-water navigation works is defined as adequate provision for navigation requiring a controlling channel depth of 27 feet.

Prior to the passing of legislation enabling Seaway construction to proceed, the International Joint Commission, created by the United States - Canada

Boundary Waters Treaty of 1909 to maintain jurisdiction over all matters pertaining to boundary waters, approved an application from the governments of the United States and Canada for the construction of power facilities in the International Section. The Power Authority of the State of New York, under license from the Federal Power Commission, and the Hydro-Electric Power Commission of Ontario have been designated as the power developing entities.

All-Canadian Section

The development of the Soulanges or the Beauharnois section of the river, which section is wholly in Canada, had its beginning in 1929. In that year, the Beauharnois Light, Heat and Power Company was authorized by Order-in-Council to divert water for power purposes from Lake St. Francis. In authorizing this diversion, the power company was required to accept conditions which respected the paramount rights of navigation on the St. Lawrence River which included provision by the power company of a navigation channel along the north bank of the power canal of adequate dimensions for Seaway traffic. The Beauharnois Power development is now operated by the Quebec-Hydro Electric Commission and this condition has been satisfied. Construction of locks by the Seaway Authority at the downstream end of the Beauharnois Canal will open the Soulanges Section for Seaway traffic.

The plan of improvement of the Lachine Section, which is the portion of the St. Lawrence River in the vicinity of Montreal, stems from the report of the Board of Engineers on the Lachine Section of October 1948. Several proposals, including those for both navigation and power and navigation alone were considered by the Board. Although the Board recognized the advantages to both navigation and power of a combined development, it has been necessary to proceed with a plan for navigation alone as it has not been considered appropriate to develop the Lachine Section for power at this time.

Basic Criteria

The basic criteria for the Seaway project canal and channel design have been developed from the report of the Joint Board of Engineers on The St. Lawrence Seaway Project, dated December 16, 1926. The principles established in the report have been rigorously followed, with the result that the Seaway canals and channels have been designed to provide for the fullest ultimate development of the navigable capacity of the waterway commensurate with cost. Every effort has been made to provide the minimum of canal navigation and the maximum amount of open river navigation. For the initial improvement of the river certain maximum standards, which are discussed in the following pages, have been adopted. However, the Seaway is being so constructed that the navigation facilities may be enlarged at minimum cost when traffic has increased to the extent that further improvements are justified. Operating experience on the Great Lakes connecting channels, the Welland Canal and the St. Lawrence River Ship Channel, has indicated the adequacy of the criteria for canal and channel design.

The United States and Canadian legislation, authorizing Seaway construction, specified that channels with 27 foot controlling depth will be provided. Accordingly, the Seaway canals and channels are being excavated to provide 27 feet of water above channel grade under any condition of operation.

Minimum Water Surface Profiles

Channel depths are being provided with reference to the minimum water levels that will obtain in the various Seaway reaches during operation of the project. A basic study, therefore, has been the determination of the minimum water surface profile, having regard to the plan for river control that will go into effect when the project has been completed and to the Seaway and power operating plans. The whole of the International Rapids Section will be greatly changed as a result of the construction of the power project. Not only will the present river be flooded out and lake-like reaches be formed, but the operation of the power project requires the establishment of a plan of regulation of outflows from Lake Ontario.

The International Joint Commission Order of Approval for the Power Project, dated October 1952, specified that a plan of regulation of Lake Ontario, known as Method No. 5, would be used in conjunction with the operation of the power project. While Method No. 5 specified criteria of regulation accepted as being appropriate at the time of the issuance of the International Joint Commission Order, it developed subsequently that errors, made apparent from a re-evaluation of basic hydraulic data, precluded the use of Method No. 5 as a satisfactory plan for Lake Ontario regulation. At about this time it became evident also that a revision of the plan of regulation could bring about a more beneficial range of stage on Lake Ontario in the interests of lake-front property owners, while at the same time providing adequately for St. Lawrence River navigation, power and downstream interests. Under the direction of the International Joint Commission, the International Lake Ontario Board of Engineers produced a plan of regulation to replace Method No. 5. This plan, referred to as Regulation Plan 12-A-9, was accepted by the International Joint Commission and recommended to the governments as being satisfactory for the purpose of calculating critical profiles and the design of channel excavations in the International Section. The significant feature of this plan from point of view of navigation is that the level of Lake Ontario will not fall below elevation 244.0, the presently accepted low water datum during the navigation season; also, the outflows from Lake Ontario will be such that the flow throughout the St. Lawrence River will not be less than the minimum experienced in the past 96 years. The recommendations of the International Joint Commission have been adopted by the governments of the United States and Canada, and the Order of Approval for the Power Project has been amended accordingly. Regulation studies are continuing in order to provide adjustments to give additional benefits to all interests concerned. However, the amendment to the Order of Approval includes the requirement that adjustments do not produce a minimum water surface profile any more severe than that established by Regulation Plan 12-A-9. Having established the sequence of Lake Ontario levels and outflows that will occur during the operation of the Seaway and Power Projects, it was then necessary to apply these circumstances to the changed conditions in order to determine precisely the minimum water surface profile.

In the International Section, the minimum water profile is dependent upon the outflow from Lake Ontario, the Lake Ontario level at which it occurs and upon the additional factor of the forebay elevation at the Barnhart Island Powerhouse. Various combinations of all three factors were considered and by means of backwater calculations, and to a considerable extent by hydraulic model studies, the minimum water surface was determined. As may be

expected, the minimum water surface elevations at all points in the International Section do not occur under the same combination of Lake Ontario level and outflow, and powerhouse forebay elevation. The profile, therefore, is actually formed by a line joining the minimum elevations that will occur at points along the section during the operation of the project.

Downstream from the powerhouse through the all-Canadian Section to Montreal, the Seaway project is being constructed to conform to the natural range of stage. For this part of the Seaway it has been necessary to consider Lake Ontario regulation only with respect to its influence on the minimum river discharges. Application of the minimum discharge to stage-discharges relationships determined from a correlation of recorded data has provided the minimum water surface profile. Since, as has been noted, the regulation plans will recognize that the Lake Ontario outflow should be no less than the minimum recorded, the minimum water surface profile should be no more severe than that of the past.

Lake Ontario regulation plans developed up to the present, use as a routing period either a month or a half-month, and it has been necessary to apply to the low-water profile derived from the application of the regulation plan an allowance to take into account the variations from the monthly or half-monthly means. Under natural conditions, it has been observed that the Lake Ontario level is influenced appreciably by wind and barometric pressure differentials over the lake surface. These phenomena propagate waves which pass down the river with the characteristics of flood waves. The behavior of these waves under natural circumstances is well defined by the records of water surface elevations maintained for many years at numerous recording stations. In sections of the river such as the International Section where considerable flooding will take place when the pool has been raised, the characteristics of these waves will be altered. Studies have been carried out independently by the United States Lake Survey and the St. Lawrence Seaway Authority to determine the magnitude of variations of this nature.

Initially, the design of navigation channels and canals in rock provided 3 feet additional depth allowance to clear rock fragments which might move into the channel during navigation. However, maintenance records of existing ship channels in rock indicate that a 2 foot allowance would be adequate. Final designs provided an additional depth of 2.5 feet above Iroquois Dam and 1.5 feet between Iroquois Dam and the Barnhart Powerhouse, which included considerations for short period variations or negative surges, as well as added depth for rock sections. The channels and canals between Iroquois Dam and the Powerhouse will be in earth material.

Downstream from the Barnhart Island Powerhouse, through the all-Canadian section of the river to Montreal Harbor, a similar procedure was used in the establishment of a minimum water surface profile. The process in this part of the river was simplified somewhat by the fact that the Seaway construction will not affect present conditions and adequate records were available from which to determine the conditions which will prevail during the operation of the project. The minimum water surface elevations were first determined for this section by the application of minimum discharges under Lake Ontario regulations to the existing stage discharge relationship. The 2 feet additional depth in rock plus an amount determined from an analysis of the records for variations from the monthly mean elevations or negative surges was then added to the 27 foot controlling depth in order to establish a channel grade elevation.

It is believed that the studies of minimum water surface elevations have resulted in a selection of grade elevations in improved sections of the river and in navigation canals which will eliminate the probability of any particular part of the Seaway becoming a ruling section.

Canal and Channel Widths

The report of the Joint Board of Engineers on St. Lawrence Waterways Project, dated November 16, 1926, Part 1, Section 11, specifies with respect to channel width as follows: "Channels for navigation have a minimum width of 450 feet, except in canal sections, where they have a bottom width of 200 feet (at 25 foot depth). Where advisable, open channels are widened at bends and to reduce cross-currents, and both widened and deepened as required to afford suitable current velocities for navigation. The minimum radius of curvature of the channels is 5,000 feet."

On the basis of these standards, the 1926 Joint Board of Engineers recommended a project which included navigation channels not less than 200 feet in bottom width when flanked by two embankments, and not less than 300 feet when flanked by one embankment, and not less than 450 feet when both sides of the channel are submerged.

In designing the Seaway channels, the same side slopes and widths at the water line were retained as in the 1926 Report, and these, together with the increase in depth of Seaway channels to 27 feet has resulted in channel bottom widths eight feet less than those suggested by the 1926 Report.

In giving consideration to the adequacy of the adopted channel widths for modern shipping, the Seaway entities were able to look to the experience gained in channels and canals of similar dimensions in the Great Lakes Connecting Channels, the Welland Ship Canal, and in the St. Lawrence River Ship Channel. These navigation facilities are in use by a large volume of shipping of the type that will use the Seaway, with relatively few delays or serious accidents. As mentioned previously, these standards for channel widths are considered to be minima, and channels are so located that they may be increased in size according to future traffic demands.

Channel Alignment

Channel alignment throughout the Seaway has been selected to provide a minimum of bends and to avoid where possible adverse cross-currents. Particular attention has been given to points where vessels enter or leave the Seaway locks and other confining sections.

Experience has shown that navigators prefer to negotiate bends defined by tangents rather than curves. The Seaway entities, therefore, adopted criteria to define the channel limits at bends as follows: "At bends, the outside limits of the channels shall be excavated to straight line tangents and chords so that the channel may be properly buoyed in accordance with the following specifications: Bends shall be widened on the inside at the rate of ten feet for each degree of deflection angle with the widening being measured along the bisector of the internal angle of these bends from the intersection of the two inside channel line tangents, and with the widened channel line at right-angles to the bisector." Figure 3A illustrates these provisions at channel bends.

Current Velocities

The plan presented by the 1926 Report of the Joint Board of Engineers permitted maximum current velocities in navigation channels somewhat less than 5 feet per second. In reviewing this requirement, the Seaway entities concluded that current velocities should not exceed 4 feet per second, and they were supported in this conclusion by the requirement contained in the International Joint Commission Order of Approval for the Power Project, which specified that current velocities in any cross-section of the river between Chimney Point and Lotus Island to be used for navigation should not exceed 4.0 feet per second at any time during the navigation season. Accordingly, channel enlargements have been designed with this requirement in mind. Actually, current velocities in river channels generally will be much less than 4 feet per second, and in the canal sections current velocities will be practically zero.

The foregoing criteria on navigation channel design have been applied to each part of the Seaway, with particular reference to the hydraulic characteristics of that particular reach. In the final analysis, the economic factor was also given very careful consideration, but was not accepted as an overriding factor. The net result will be channels and canals of standards superior to those defined by the criteria.

Canal and Channel Design

Welland Ship Canal

Lake Erie, the upper limit of the Seaway as defined in the legislation, is at elevation 572 and the descent by navigation of 326 feet to Lake Ontario, at elevation 246, is through the Welland Ship Canal, a navigation system constructed by Canada in 1932. With the exception of the 25 foot controlling depth in the canal reaches, the Welland Ship Canal meets Seaway standards of the present time. Seaway construction at the Welland, therefore, is limited to the improvement of the canal reaches by deepening so as to provide a controlling depth of 27 feet, and with some minor widening to provide additional passing places.

Thousand Islands Section

From Point Weller, Ontario, at the lower entrance of the Welland Ship Canal, across Lake Ontario to the head of the St. Lawrence River at Tibbetts Point, New York, the vessel track is 160 miles. In the first 68 miles of the St. Lawrence River downstream from Tibbetts Point, New York to Chimney Point, New York, the river falls only about one foot, and this section may be considered to be an extension of Lake Ontario. Here the river is slow-moving, generally deep, and is from one to four miles in width. Its many islands and shoals has given rise to its name—the "Thousand Islands Section."

The islands and shoals are composed largely of precambrian granite, granite gneiss, and quartzite. Conglomerate and sandstone outcrops occur at Wellesley Island and on the mainland in this vicinity. Over-burden encountered over some of the shoal areas is in thin layers, and consists of sand and gravel.

Construction of the Seaway channel through the Thousand Islands Section consists essentially of marine excavation, principally in rock, to provide a

controlling channel depth of 27 feet. Although the work is difficult and expensive, the quantity of rock to be removed is relatively small. In fact, the channel through the Thousand Islands Section is presently in use by the largest upper lake bulk carriers, which bring grain cargos down to the Canadian Government elevator near Prescott, Ontario, for transhipment.

The construction of the Seaway channel through the Thousand Islands Section is being undertaken jointly by the Seaway Development Corporation and the Seaway Authority, with each doing the portion of the work in its own territory. By agreement between the two entities, all of the basic field surveys were undertaken by the Corps of Engineers, United States Army, design and construction agent for the Saint Lawrence Seaway Development Corporation. These field surveys included bar and wire sweeping and detailed probing in the shoal areas, and also core drilling for subsurface exploration.

Improvement of the Thousand Islands Section for navigation began in 1890 under the Corps of Engineers Rivers and Harbors Act, and the part of the channel in United States waters was improved to a depth of about 18 feet. A comprehensive improvement was undertaken by the United States and Canada beginning in 1930, which resulted in the channel as it presently exists and which provides a minimum depth of 27 feet when Lake Ontario is at elevation 244.5. Channel width in restricted sections is somewhat less than 450 feet. On the basis of the minimum water surface profile and additional depth allowance adopted by the Seaway, the existing channel grade will be lowered an average of 3.0 feet. Both Public Law 358 and the St. Lawrence Seaway Authority Act provide for the improvement of the Thousand Islands Section as part of the over-all Seaway project. The design criteria are those which have been previously described, and the improvement now in progress will result in a controlling channel depth of 27 feet, and bottom width of at least 450 feet.

In establishing the grades in the areas to be dredged, an additional 2.5 feet allowance has been made due to the fact that the channel work is essentially in rock, and to take into account short period water level fluctuations. Since the discharges and water levels of the Thousand Islands Section will be affected directly by the plan adopted for regulation of Lake Ontario, the circumstances obtaining under the presently accepted method, which is Plan 12-A-9, have been considered in establishing the minimum water surface profile. However, the improvements through the Thousand Islands Section will have no practical effect on its present hydraulic characteristics and recorded circumstances are the chief source of data.

International Rapids Section

The 44 mile reach of the St. Lawrence River downstrean from Chimney Point, New York to St. Regis, New York, as in the case of the Thousand Islands Section, is partly in Canada and partly in the United States. Through the section, the river falls approximately 92 feet, with the greater portion of the fall concentrated at the Galop Rapids, a few miles downstream from Chimney Point, New York, at the Rapids Plat in the vicinity of Morrisburg, Ontario, and at the Long Sault Rapids. The Galop Rapids is actually the natural control for Lake Ontario, consequently, any improvement in the river in this area will have a very definite effect upon the outflows and levels in Lake Ontario.

This 44 mile reach is commonly referred to as the "International Rapids Section" and it is one of the sections in the St. Lawrence River in which multipurpose development is taking place. Development of the river for power by

the Power Authority of the State of New York and the Hydro-Electric Power Commission of Ontario includes the construction of a powerhouse at the downstream end of Barnhart Island, a spillway dam at the Long Sault Rapids and a control dam at Iroquois. When the pool above the powerhouse has been raised in 1958, the present canal system upstream of Cornwall, Ontario, will be inundated. The new Seaway features being constructed to replace the old system and to provide a new and up-to-date facility consist of two locks and connecting canals in the United States to bypass the powerhouse and spillway dam, and a lock in Canada to bypass the control dam at Iroquois. Improvement of the river reaches by channel enlargement, together with the creation of the lake-like reach above the powerhouse, provide an opportunity to develop Seaway channels of excellent alignment and of adequate width and depth for Seaway traffic.

Galop Rapids to Long Sault Canal

The navigation channel downstream from Chimney Point passes from a wide and deep section of the river through a series of confining channels developed in conjunction with the power development. The first of these is at Galop Island (See Figs. 2 and 3), where, in accordance with the international Joint Commission Order of Approval, extensive channel enlargement is being carried out by the Power entities. The channel enlargements between Chimney Point and Lotus Island, some ten miles downstream, are specified in the Order of Approval to insure that the average current velocity in any cross-section to be used by navigation will not exceed 4.0 feet per second at any time during the navigation season.

To design this feature and to verify that the specifications of the Order of Approval have been met, the Power entities carried out an extensive model testing program at the Hydraulic Laboratory of the Hydro-Electric Power Commission of Ontario. The Joint Board of Engineers has kept the Seaway entities informed continuously as to the progress of the model studies, and the opportunity was thus afforded to offer comments and suggestions which did much to insure the most satisfactory design of that part of the channel enlargement intended to be principally a navigation feature.

The model study of the Galop Rapids reach was but a small part of the extensive model testing program undertaken by the Power and Seaway entities as all the important sections of the river in the International Rapids Section were carefully studied by the use of hydraulic models.

The International Joint Commission Order of Approval stipulated also that the reaches of the river from below Lotus Island to above Iroquois Dam and from Point Three Points to below Ogden Island are to be designed to provide a maximum current velocity during the ice-forming period not exceeding 2.25 feet per second. The design required to satisfy this requirement was also carefully studied on the hydraulic models at the hydraulic laboratory of the Hydro-Electric Power Commission of Ontario. As a consequence of the enlargement of this part of the river in order to maintain velocities at or below 2.25 feet per second during the ice-forming period, current velocities in navigation channels will not exceed 4.0 feet per second during the navigation season. As in the case of the design of the Galop Rapids reach, the Joint Board of Engineers solicited comments from the Seaway entities concerning the design of the channels which were intended primarily as ice cuts. Here again, the Seaway entities were able to suggest and recommend minor modifications

which will result in more satisfactory circumstances for shipping.

The degree of coordination required among the interests involved in the development of the International Section will be appreciated when it is realized that construction of many of the features was actually in progress before design had been fully completed. For example, the Seaway locks were under construction before the final design of the Galop Rapids reach had been approved by the Joint Board of Engineers and each modification of the original and basic design affected the water levels which would be experienced at the lock sites during operation. The matter was further complicated by the fact that regulation plans for Lake Ontario, as mentioned previously, were being carried out concurrently with construction. The necessary coordination has been accomplished and no major changes were necessary in any plans once they were accepted as final.

The plan approved by the Joint Board of Engineers for channel work undertaken by the Power entities in conjunction with Regulation Plan 12-A-9 produced critical water surface profiles under the following conditions:

a) From Chimney Point to Iroquois Dam - Lake Ontario Water Level 244.95 - Lake Ontario outflow 277,000 c.f.s.

b) Iroquois Dam to Point Three Points - Powerhouse forebay elevation 238.0 - Lake Ontario outflow 180,000 c.f.s.

c) Point Three Points to Eisenhower Lock - Lake Ontario Water Level 245.56 - Lake Ontario outflow 296,000 c.f.s.

The condition producing maximum current velocities in navigation channels was found to be with Lake Ontario Water Level at 245.56 and with outflow 296,000 c.f.s.

From Morrisburg downstream to the head of the Long Sault Canal, no major improvements are contemplated. However, further surveys in this section, including sweeping after the pool has been raised, may indicate a minor amount of marine excavation or sailing line relocation. Current velocities in this section of the river will be very low, and a channel alignment most satisfactory to navigation may be provided. Channel widths are at least 600 feet in the river reach between Chimney Point and the head of the Long Sault Canal.

Iroquois Lock and Canal

At Iroquois, the Seaway Authority is constructing a lock across Iroquois Point to bypass the control dam of the Power entities. The approaches to this lock were also given careful study on the hydraulic models to determine whether or not adverse cross-currents would exist during operation at the upper and lower entrances. Particular attention was given to the operation of the Iroquois Dam insofar as it may affect navigation at this point. The final design of the upper entrance to the lock was coordinated with the enlargements being carried out by the Power entities at Sparrowhawk Point and at Toussaints Island.

Long Sault Canal and Locks

The Long Sault Canal and the Eisenhower and Grasse River Locks are Seaway features being constructed by the Saint Lawrence Seaway Development Corporation to bypass the Long Sault Dam and Barnhart Island

Powerhouse. The total average fall overcome by these features is about 85 feet.

Detailed studies of the Long Sault Canal were made in connection with the 1942 report of the United States Corps of Engineers and, in addition to the accepted criteria for canal and channel design, certain other aspects unique to this feature were given a great deal of attention. For example, an entrance had to be located that could be found without difficulty after traversing, possibly in fog or bad weather, approximately 17 miles of open water from Iroquois. In all, six separate locations were studied, and the canal as now being constructed has been accepted by the Seaway Development Corporation as providing the most satisfactory entrance.

The incidence of adverse currents during operation was another matter investigated, and for this purpose the Seaway Development Corporation had a series of model tests undertaken at the Hydraulic Laboratory of the Hydro-Electric Power Commission of Ontario.

The Long Sault Canal alignment selected cuts inland at Richards Point and extends almost due east to the mouth of the Grasse River. When the pool has been raised, vessels using the Long Sault Canal will pass above natural and deep channels which are now part of the river bed and the most important of these, where the possibility of cross-currents might exist, is at the Big Sny. When subjected to model testing the cross-currents at Big Sny were found to be almost 90° to the navigation channel and in excess of 4 feet per second. To limit the adverse effects, additional enlargements were made across Croil Island to alter the flow pattern.

Both banks throughout the canal will be submerged except for short reaches and, in accordance with the basic criteria, the channel bottom width is 442 feet. The grade elevation of the canal upstream from the Eisenhower Lock was selected after consideration of the manner in which the pool will be regulated during the operation of the power project. The minimum water surface elevation at the upper end of the canal was calculated to be 236.0. Then 27 feet, the controlling channel depth, plus 1.5 feet, the accepted allowance for negative surges in the power pool, were deducted from this elevation to give a grade elevation of 207.5. To allow for surges originating from the operation of the Eisenhower Lock, the grade from Hopson's Bay to the Eisenhower Lock has been set at elevation 207.0.

The water surface in the reach between the Eisenhower and Grasse River Locks is to be controlled at elevation 200.0. Computations revealed that the filling of Grasse River Lock from the intermediate pool indicated a possible negative surge in the intermediate pool of approximately 1.8 feet. This allowance was increased to 2.5 feet to include wind effects, thus the grade of the channel through the intermediate pool has been fixed at elevation 170.5.

The entire Long Sault Canal is in glacial till and marine clay, which eliminates any need for the additional depth allowance as in the case of rock cuts. However, one foot overdepth is being allowed to take care of inaccuracies in excavation. The side slopes are no steeper than 2 horizontal to 1 vertical, and are flattened as determined by the circular arc method of analysis, depending upon the nature and depth of the cut.

Cornwall Island Reach

Downstream from the Barnhart Island Powerhouse, the St. Lawrence River divides into two channels at Cornwall Island. The Seaway channel just below the Grasse River Lock passes through the South Cornwall Channel to the

easterly end of Cornwall Island, thence north of St. Regis Island into Lake St. Francis.

In its natural condition the South Cornwall Channel is comparatively shallow with restricted sections of fast water with current velocities as high as 10 feet per second. The problem of providing a satisfactory navigation channel through this section is further complicated by the discharge at high velocity through Polleys Gut into the South Channel of approximately two-thirds of the total river flow. Immediately downstream from the Grasse River Lock entrance this flow must make a 90° turn, and this condition, if not modified, would produce cross-currents much too severe for navigation.

To study adequately the design of the channels in the Cornwall Island reach, both the Saint Lawrence Seaway Development Corporation and the Seaway Authority resorted to model studies. The first model was constructed by the Seaway Authority at the National Research Council laboratory at Ottawa for the purpose of studying the effects of both navigation and power improvements. This model was constructed to a distorted scale ratio of 1:480 horizontal and 1:96 vertical, and covered the reach of the river from the tailrace of the Barnhart Island Powerhouse to below St. Regis Island. Although it had been contemplated that this model would be used also to test the proposals for navigation improvements in the South Cornwall channel, it was deemed advisable to construct an undistorted model representing this section of the river in order to insure reliable results from tests of a channel containing abrupt changes in flow direction. Accordingly, a model at a scale of 1 to 100 was constructed at the Waterways Experiment Station at Vicksburg, Miss. This model covered the reach from the tailrace of the Barnhart Island Powerhouse downstream to a point in both the north and south channels, opposite Raquette Point.

As the work load on the models increased, another model was constructed at Vicksburg to a scale of 1 to 300 horizontal and 1 to 100 vertical, which reproduced the same reach of the river as the model at the National Research Council laboratory.

Testing programs on all three models produced a channel improvement feature which is now under construction. It consists essentially of enlargements of the river cross-section through the south channel, which, together with a compensatory dredging feature in the north channel, produces a reduction in current velocities to a maximum of 4.0 feet per second, as required for navigation. Channel grades for the enlargement in the south channel are generally lower than those required to give channels of 27 foot controlling depth, being at elevation 120.5 in the sections where cross-sections have been increased for current velocity reduction. To offset the effect of the high velocity discharge from Polleys Gut, training dikes have been designed to direct the flow in such a manner that it enters the navigation channel gradually, thus eliminating adverse cross-currents.

Control works at the outlet of Lake St. Francis would permit the maintenance of a wide range of lake stages; however, in the design of the Cornwall Island channels Lake St. Francis was assumed to be controlled as in the stage of nature. With Lake St. Francis so controlled and with the minimum Lake Ontario outflow as provided in presently approved Lake Ontario regulation plans, the minimum water surface elevation was determined to be 150.5. This determination took into account the effect of wind and other factors which might cause short period variations. Channel grades have been established

accordingly at elevation 123.5, except where an increase in cross-section is necessary to reduce velocities as stated above.[1]

Lake St. Francis

Downstream from the International Rapids Section, the St. Lawrence River lies wholly within Canada, and improvements in the river downstream to Montreal are being provided by the St. Lawrence Seaway Authority.

The navigation channel improvements in the South Cornwall Channel are continuous across the International Boundary into the Lake St. Francis Section at the downstream end of Cornwall Island. Below Cornwall Island, the navigation course crosses Lake St. Francis for a distance of 31 miles to the head of the Beauharnois Power Canal. Channel improvements being undertaken by the Authority consist of marine excavation at shoal areas in the lake to give a channel with a minimum width of 450 feet and 27 feet controlling depth, and the length of the conforming channel is only 5.5 miles. Current velocities present no difficulty in Lake St. Francis, consequently, no enlargements are being undertaken expressly for this purpose.

As in the Cornwall area, the elevation of channel grades through Lake St. Francis have been established on the basis of lake levels controlled as in the state of nature, with the minimum regulated outflow from Lake Ontario. The selected grade elevation is 123.0.

Beauharnois Channel and Locks

Downstream from the Lake St. Francis section is the portion of the river referred to as the Soulanges or Beauharnois Section. In this section, the improvement of the river is in the form of a multipurpose development for both navigation and power. It differs from the International Rapids Section in that the development of power has preceded the development of navigation.

At the downstream end of Lake St. Francis, a diversion canal was constructed in 1932 as a feature of a power development, through which water is diverted from Lake St. Francis to a powerhouse located on Lake St. Louis, fifteen miles downstream. Extensions to the powerhouse have increased the installed capacity to its present 1,425,000 h.p., and a further extension now under construction will increase the rating to 2,160,000 h.p. Progressive enlargements of the diversion canal to accommodate the resulting increased discharges have produced a broad, deep channel through which a large percentage of the total river flow now is diverted.

In authorizing the diversions of water for power purposes at Beauharnois, Canada specified certain conditions which would enable the power canal to be used ultimately as part of the Seaway system. Specifically, the power development company, which at the present time is the Quebec Hydro-Electric Commission, has been required to maintain the canal to give a clear width of 600 feet on the bottom, a depth of 27 feet at low stage, and to such cross-section as to produce average velocities of not more than 2.25 feet per second under any condition of operation. Thus, by legislation passed in 1932, the Government of Canada made provision for an important feature of the Seaway project

1. Elevations upstream of Cornwall Island are referred to in the 1935 U.S.L.S. datum. Elevations in the all-Canadian section refer to the Geodetic Survey of Canada datum. At Cornwall, 1935 U.S.L.S. elevations = G.S.C. elevations plus 0.5 feet.

at the Beauharnois Section. The two Seaway locks now under construction at the lower end of the canal to overcome the 84 foot fall between Lake St. Francis and Lake St. Louis will complete the Seaway construction at Beauharnois.

The reach between the upper and lower Beauharnois locks will be a regulated pool similar to that between the Eisenhower and Grasse River Locks. It will be controlled to an average elevation of 110.0, but to provide for additional depth in rock, negative surges, and variations due to lockage operations, the channel grade has been established at elevation 76.0. Construction of this channel is through extremely resistant Potsdam sandstone, and excavation is being done in the dry.

Lake St. Louis

The navigation channel downstream from the lower Beauharnois lock extends for ten miles through Lake St. Louis to above St. Nicholas Island, where the channel comes close to the south shore and passes inland at Caughnawaga. Improvement through Lake St. Louis consists of marine excavation to provide a navigation channel with 600 feet minimum width.

Elevation 38.0 has been selected as the grade elevation for the channel through Lake St. Louis, and this was determined from a minimum water surface profile that would result from the natural stage of the lake, equivalent to the minimum outflow under Lake Ontario regulation plus flow from the Ottawa River. Studies in this connection have taken into account the influence of discharges of the Ottawa River, part of which enters the St. Lawrence River at Lake St. Louis. The remaining Seaway channel downstream from Lake St. Louis to Montreal Harbor is in the part of the river referred to as the Lachine Section, so-called because of the Lachine Rapids, the principal feature of the river in this area. From Caughnawaga to Montreal the normal fall during the navigation season is about 47 feet, and this fall will be overcome by Seaway facilities consisting of two locks, one at Cote Ste. Catherine and the other at St. Lambert, together with connecting channels and canals.

Caughnawaga - Cote Ste. Catherine Canal

The overland channel begins at Caughnawaga on the south side of the river and extends for eight miles to the Cote Ste. Catherine Lock. This channel, which is being excavated in the dry, will be of 250 feet minimum width, and as in the Lake St. Louis channel, the grade elevation is at 38.0. Canal construction is principally in rock which varies from Trenton limestone to Utica shale, although in some parts of this reach the channel is in marine clay with rock below the finished channel grade. Side slopes in rock sections are 1.5 horizontal to 1 vertical, and are flattened elsewhere in the softer material as required to assure stability.

The interesting fact relating to the Caughnawaga - Cote Ste. Catherine Canal is that it has been designed to be used if required in the future in conjunction with the development of the Lachine Section for power. Its function at that time will be as a bypass channel through which up to 40,000 cubic feet per second may be passed during the ice-forming period in order to facilitate the formation of an ice cover over the forebay of a powerhouse to be constructed at the foot of the Lachine Rapids.

Cote Ste. Catherine - Montreal Harbor

From the Cote Ste. Catherine Lock downstream, a distance of eight miles to the St. Lambert Lock, the navigation channel will be isolated from Laprairie Basin by a continuous dike. This will permit the pool between the locks to be controlled during the navigation season initially at elevation 35.0 and ultimately, should the need arise, at elevation 38.0. Since in this section one bank of the canal will be submerged and the other will be formed by the Laprairie Dike, the channel width is 300 feet. The grade elevation has been established at elevation 8.0. Current velocities in this portion of the canal will be practically zero, and variations from the regulated level will be insignificant, therefore, the canal will give most satisfactory conditions for Seaway navigation.

Downstream from the St. Lambert Lock, the Seaway enters Montreal Harbor through a channel leading from the lock for a distance of two miles. In this section, the canal banks rise above the water surface and the canal bottom width is 225 feet. To establish the channel grade in this section, a very careful study was required of many factors which included not only river discharges which will occur under Lake Ontario Regulation, but the progressive lowering effect on harbor levels produced by the continuous improvement in the St. Lawrence Ship Channel and in Montreal Harbor. Considering these factors, channel grade has been established at -16.0. The whole of the channel downstream from the Cote Ste. Catherine Lock is being excavated in the dry, mostly through rock. Because of this, side slopes are generally 1.5 horizontal to 1 vertical.

Transition—Existing 14 Foot Canadian Canals to New Seaway above Cornwall, Ontario

An interesting aspect of the Saint Lawrence Seaway development is the process of change-over from the present 14 foot navigation facilities to the new Seaway facilities.

In the Lachine Section, the new canals and locks are entirely separate from the present system. The same is true in the Soulanges or Beauharnois Section. Consequently, there is no problem created during the change-over. The new navigation facilities in these sections will be ready to accommodate Seaway traffic when navigation opens in 1959. Navigation aids will have been established, and shipping will be directed to the Seaway at Montreal Harbor and at Lake St. Francis, and few delays are anticipated. Parts of the old canal system will continue to function as long as industry located along the canals, particularly at Montreal and Cornwall, has need for it.

In the river between Prescott, Ontario and Cornwall the situation will be quite different. In this section the present 14 foot canal system will be flooded out when the pool upstream from the powerhouse is raised, also, shipping will be subjected to some relatively minor inconveniences during construction. The chief problem is one of raising the pool elevation and keeping to a minimum the delays to vessel movements. The pool is scheduled to be raised on July 1, 1958, which month falls in the middle of the navigation season and delays of any duration, therefore, would be undesirable.

It is proposed to handle the change-over about as follows in the International Rapids Section. By late 1956, the construction in the river in the vicinity of Iroquois had progressed to the point that required navigation to use

the canal in both directions between Iroquois and Cardinal, instead of the river below Lock 28 for downbound shipping, as was previously the practice. The Department of Transport has provided passing points and additional tie-up facilities in the canal in anticipation of an increased volume of navigation.

Shortly after April 1957, the water level above Iroquois Dam will gradually be raised and the river brought under regulation. The regulation "rule" at that time will be as in the state of nature. This means that Lake Ontario levels and outflows, while under artificial control, will be kept as under natural circumstances. The raising of the water level above Iroquois Dam will keep pace with the enlargement of the river upstream, particularly at the Galop Island cut. The canal banks of the present system will be raised where necessary to retain the water in the canal reaches at present levels. During early 1958 the Seaway lock at Iroquois may be brought into service for 14 foot navigation. At that time the water levels above the Iroquois Lock will be close to the ultimate while the water level below the lock will be at natural condition or slightly lower. This means that the lock may be required to operate at a head of up to 24 feet, and it has been designed to do so. The limitation on the use of the Iroquois Lock for 27 foot navigation during this period is the limited depth on the lower sill. As the pool above the power-house is raised, the lock will be available for vessels of full Seaway draft.

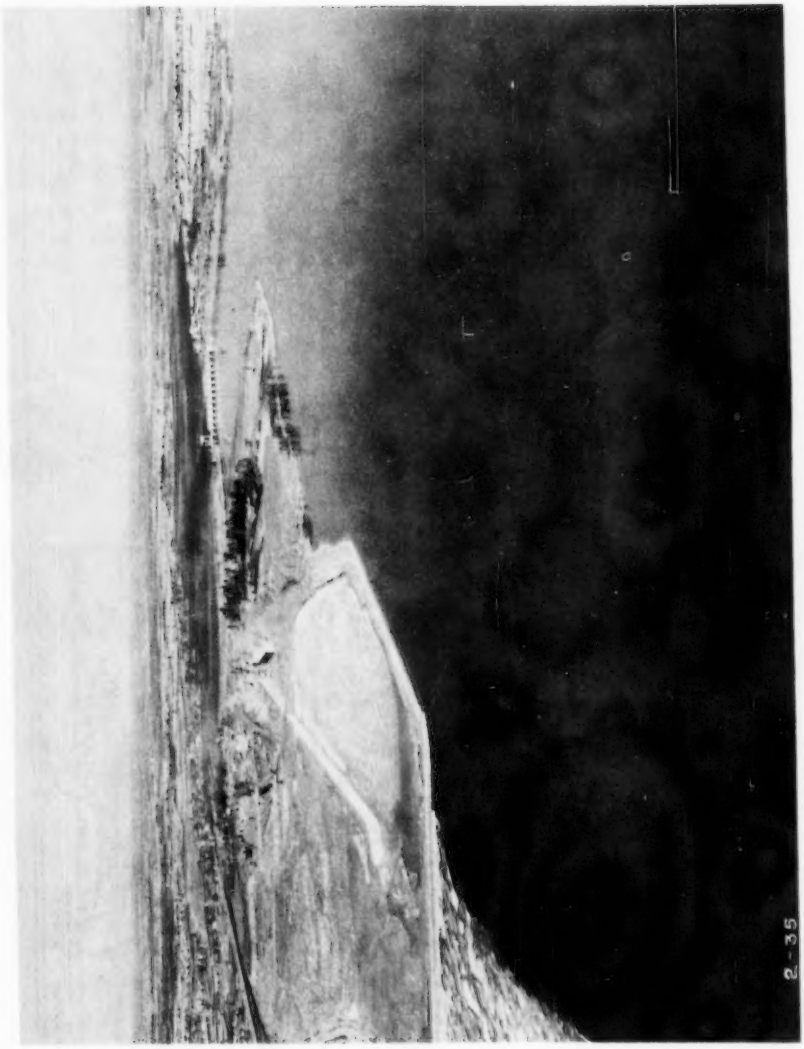
Until the pool is raised in 1958, the river and canal sections between Iroquois and Cornwall will be used as at present. A diversion canal and dike closure structure have been constructed at the north end of the Barnhart Island Powerhouse, and these facilities will be put into use in April 1957. The main dike may thus be properly constructed across the present Cornwall Canal, and when the time comes to raise the pool, stop logs may be quickly placed in the closure structure, thereby closing off the 14 foot canal. The two locks on the United States side will be ready for at least 14 foot draft shipping when closure is made and the pool raising begins.

It is essential that a continuous flow of water be maintained in the river at all times to protect the downstream power and navigation interests and it is proposed, therefore, to raise the pool by a transfer of water from Lake Ontario. The quantity of water required to fill the pool (500,000 acre feet) will not affect Lake Ontario to an appreciable extent. At the present time it appears that a rapid raising of the pool to elevation 235 will present the least difficulty to navigation, and the delay that will result will not be more than four days. This proposal for change-over procedure depends on the stage of Lake Ontario at the time the pool is to be raised.

Status of Design

The design of the Seaway canals and channels is virtually complete, and construction is well advanced in all sections. Much work remains to be done to establish navigation aids and in chart compilation in order to make the facilities being provided fully useful to navigation. All this work will be completed in time, however, for the opening of the Seaway in the International Section in 1958 for vessels of 14 foot draft, and the whole of the Seaway for 27 foot navigation in early 1959.

Eisenhower Lock and Long Sault Canal, United States Mainland



Iroquois Lock, Ontario, Canada Mainland

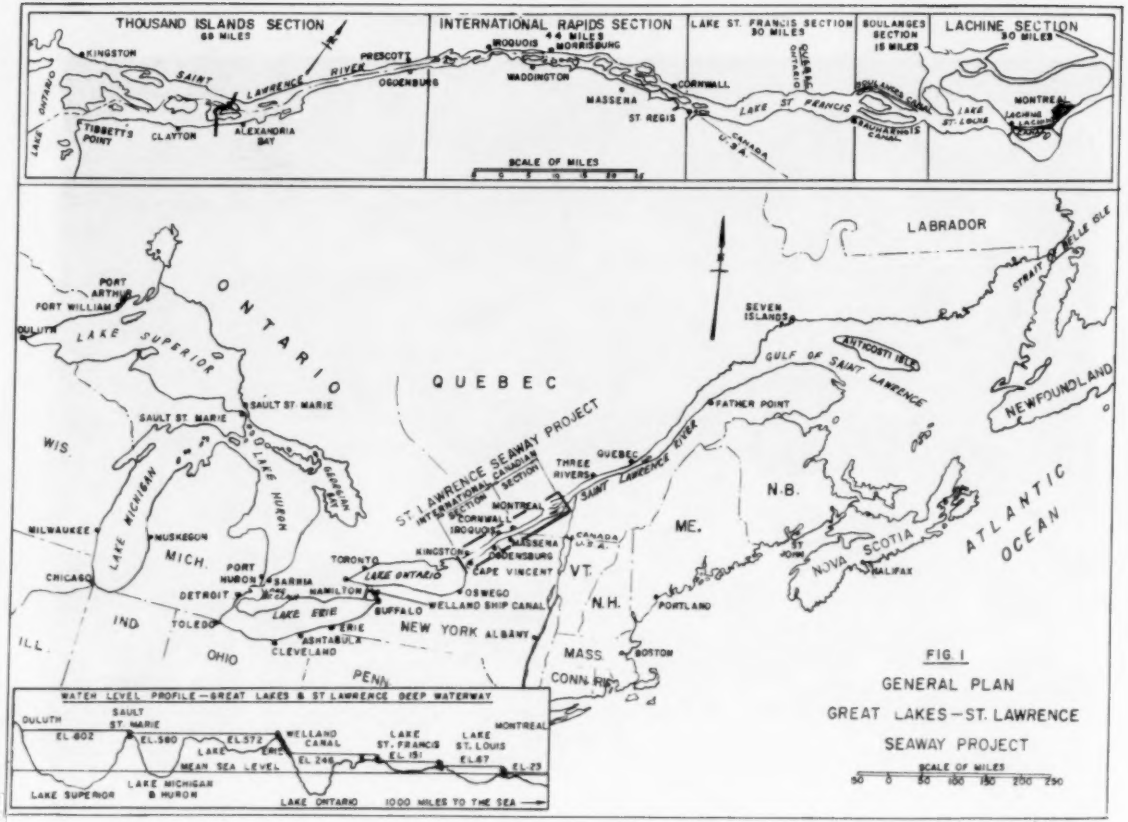

THOUSAND ISLANDS SECTION
68 MILES
INTERNATIONAL RAPIDS SECTION
44 MILES
LAKE ST. FRANCIS SECTION
30 MILES
SOULANGES SECTION
15 MILES
LACHINE SECTION
30 MILES
KINGSTON
PRESCOTT
IROQUOIS
MORRISBURG
WADDINGTON
MASSENA
CORNWALL
ST REGIS
OGDENSBURG
TIBBETT'S POINT
CLAYTON
ALEXANDRIA BAY
SAINT LAWRENCE RIVER
LAKE ONTARIO
LAKE ST. FRANCIS
CANADA U.S.A.
SCALE OF MILES
MONTREAL
LAKE ST. LOUIS
LACHINE CANAL
LABRADOR
STRAIT OF BELLE ISLE
SEVEN ISLANDS
ANTICOSTI ISLE
GULF OF SAINT LAWRENCE
GASPE POINT
NEWFOUNDLAND
ATLANTIC OCEAN
ONTARIO
QUEBEC
PORT ARTHUR
PORT WILLIAM
DULUTH
LAKE SUPERIOR
SAULT ST MARIE
WIS.
LAKE MICHIGAN
LAKE HURON
GEORGIAN BAY
MILWAUKEE
MUSKEGON
MICH.
CHICAGO
PORT HURON
SARNIA
DETROIT
TOLEDO
IND.
ILL.
OHIO
PENN.
CLEVELAND
ASHTABULA
ERIE
LAKE ERIE
BUFFALO
TORONTO
HAMILTON
LAKE ONTARIO
KINGSTON
OSWEGO
CAPE VINCENT
WELLAND SHIP CANAL
NEW YORK
ALBANY
ST. LAWRENCE SEAWAY PROJECT
INTERNATIONAL SECTION
CANADIAN SECTION
CORNWALL
IROQUOIS
MONTREAL
THREE RIVERS
QUEBEC
SAINT LAWRENCE RIVER
CANADA U.S.A.
MASSENA
OGDENSBURG
ME.
N.B.
VT.
N.H.
PORTLAND
MASS.
BOSTON
CONN. R.I.
NOVA SCOTIA
ST JOHN
HALIFAX
WATER LEVEL PROFILE—GREAT LAKES & ST LAWRENCE DEEP WATERWAY
DULUTH
SAULT ST. MARIE
MONTREAL
EL. 602
EL. 580
EL. 572
WELLAND CANAL
LAKE ST. FRANCIS
LAKE ST. LOUIS
LAKE ERIE
MEAN SEA LEVEL
EL. 246
EL. 151
EL. 87
EL. 23
LAKE SUPERIOR
LAKE MICHIGAN & HURON
LAKE ONTARIO
1000 MILES TO THE SEA
FIG. 1
GENERAL PLAN
GREAT LAKES—ST. LAWRENCE
SEAWAY PROJECT
SCALE OF MILES

FIG. 2A

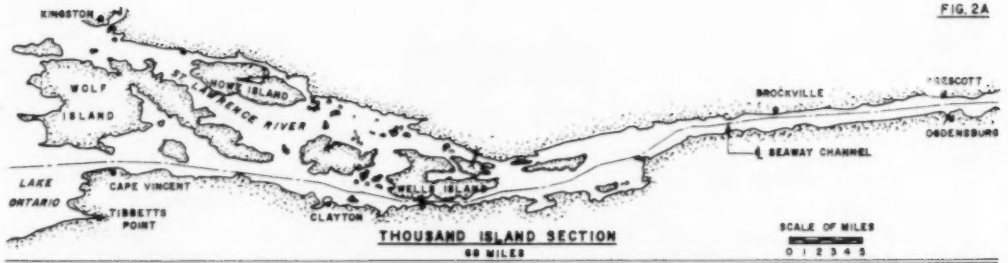


FIG. 2B

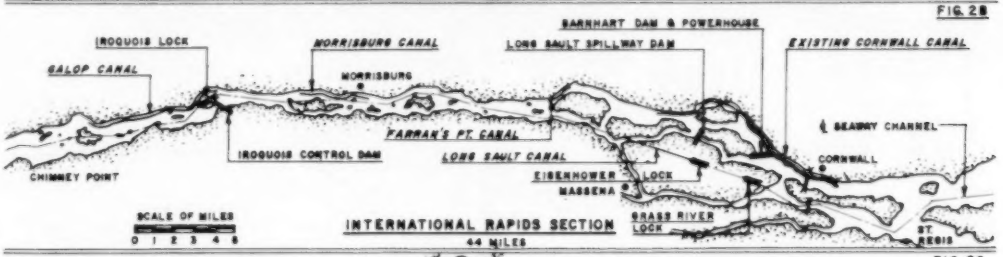


FIG. 2C

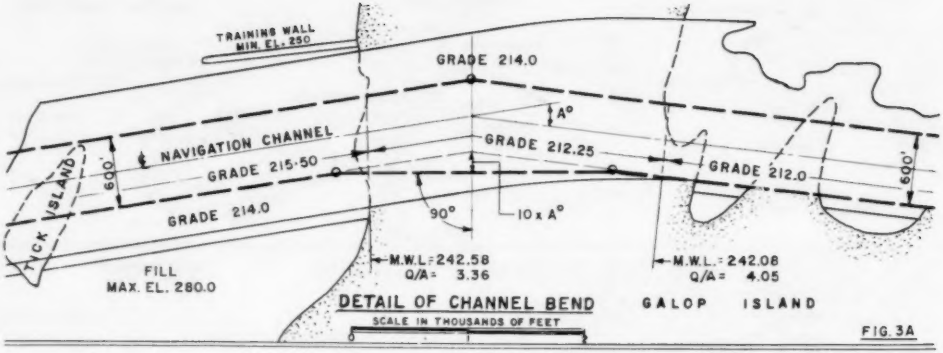

TRAINING WALL
MIN. EL. 250
GRADE 214.0
A°
NAVIGATION CHANNEL
GRADE 212.25
GRADE 215.50
GRADE 212.0
600'
600'
HICK ISLAND
GRADE 214.0
90°
10 x A°
M.W.L.=242.58
Q/A= 3.36
M.W.L.=242.08
Q/A= 4.05
FILL
MAX. EL. 280.0
DETAIL OF CHANNEL BEND
GALOP ISLAND
SCALE IN THOUSANDS OF FEET

FIG. 3A

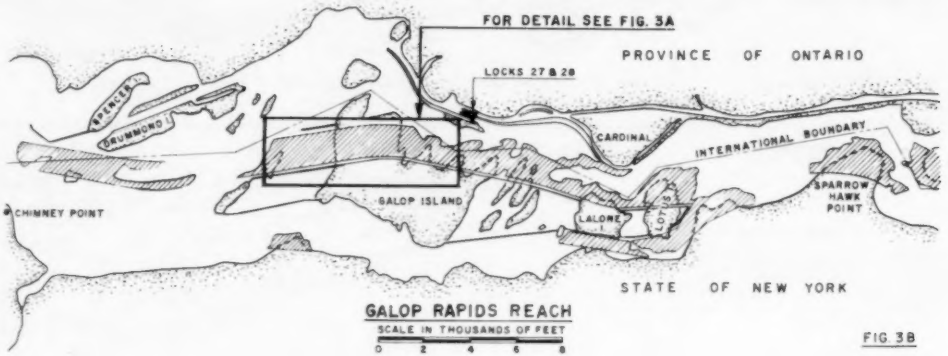

FOR DETAIL SEE FIG. 3A
PROVINCE OF ONTARIO
LOCKS 27 & 28
SPENCER
DRUMMOND
CARDINAL
INTERNATIONAL BOUNDARY
GALOP ISLAND
SPARROW HAWK POINT
CHIMNEY POINT
LALONE
LOCK 28
STATE OF NEW YORK
GALOP RAPIDS REACH
SCALE IN THOUSANDS OF FEET
0 2 4 6 8

FIG. 3B

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1523)

Page

Anchorages for Large Tainter Gates, by Alexander H. Kenigsberg. (Proc. Paper 1119. December, 1956. Prior discussion: 1381. Discussion closed.)
by Alexander H. Kenigsberg (closure) 1523-3

Rivers Under Influence of Terrestrial Rotation, by Otakar W. Kabelac. (Proc. Paper 1208. April, 1957. Prior discussion: 1381. Discussion closed.)
by Otakar W. Kabelac (closure) 1523-7

The Littoral Drift Problem at Shoreline Harbors, by J. W. Johnson. (Proc. Paper 1211. April, 1957. Prior discussion: none. Discussion closed.)
by R. Silvester 1523-13

Note: Paper 1523 is part of the copyrighted Journal of the Waterways and Harbors, Division, Proceedings of the American Society of Civil Engineers, Vol. 84, WW 1, January, 1958.

ANCHORAGES FOR LARGE TAINTER GATES[a]

Closure by Alexander H. Kenigsberg

ALEXANDER H. KENIGSBERG,* M. ASCE.—Without the benefit of drawings pertaining to the tainter type gates for the power intakes of the Macagua No. 1 hydroelectric scheme in Venezuela, it is somewhat difficult to comment on Mr. S. H. Wearne's recitation of the gate anchorages employed on this project. However, it appears from his description that the substitute arrangement, used in place of the original anchorage design which was similar to the conventional type of the paper, is not an optimum from the standpoint of transferring applied loads through constituent parts of a structural ensemble to their ultimate supports.

First, the statement that "the trunnions are mounted on plate grillages" implies that the entire hydrostatic thrust of some 300 tons on each trunnion, in place of the 600 tons given in the discussion, is transmitted to the grillages through bolts, in shear and bending. Second, "the heavy plates lying in the plane of the buttress faces" bespeaks that a considerable portion of the thrust is applied thru the edges of the plates by bearing on the outer facing of the concrete buttresses. Third, "the grillages comprise a pair of heavy plates coupled by bolts. . . . distributing the force to reinforcement" indicates the absence of a solid metallic member between these plates and consequent transfer of the balance of the water thrust through the bolts by bearing on the masonry, necessarily prior to the interception of the force by the reinforcement.

By inspection of Figs. 4 and 5 of the paper, it can be noted that both the conventional and improved single girder anchorages transmit the water load on the gates directly to integral anchorage frames and, through their embedment, well into the mass of the spillway masonry. Just as the basic designs of the large tainter gates proper, referred to in Mr. Buzzell's paper, both types of anchorages were developed and applied to several projects by the Nashville District, Corps of Engineers, and have been functioning most successfully for a good many years.

In answer to Mr. Wearne's query the writer would like to submit a few reasons for the continued use of the term "tainter" in preference to "radial" gates:

1. It is a tribute due Mr. J. B. Tainter in recognition of his valuable contribution in the field of hydraulic gates that is still gaining in general acceptance over other types. As is more frequently found in the sciences and other professions, it is thought that a wider practice of perpetuating

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a. Proc. Paper 1119, December, 1956, by Alexander H. Kenigsberg.

* Civ. and Structural Engr., Nashville, Tenn.

engineers' names by their inventions and original developments would redound to the advancement of engineering, and the profession as well, a great deal more than by the employment of descriptive terms alone.

2. Other types of hydraulic gates, such as some flash board, bear trap and drum gates, could more properly be labeled as radial gates since they function through the revolution of radial planes about an axis, whereas, in the tainter type, the gate proper is a partial cylindrical surface whose end arms alone are radial. There is already frequent misunderstanding and misapplication of gate terms to entirely distinct species, such as roller and rolling type gates.

3. For optimum hydraulic operation of navigation locks, in providing against the effects of surging and air turbulence, the flow of water in the culverts is usually regulated by reverse tainter gates. Also, when compound crest gates are involved for control of spillway flows, arrangements of opposed tainter gates have been considered. In such cases the use of "reverse radial" would seem to be an inadequate term, whereas "reverse tainter" is an accepted clear-cut concept for conveying their true functioning. Thus, it would have been helpful if the power intake gates cited by Mr. Wearne were referred to as "reverse tainters" since it appears that the water pressure is applied from their trunnions toward the inside of the cylindrical surfaces.

Parallel Arms Tainter Gates

Except for a very brief reference, discussion of anchorages for parallel arms gates was spaced out from the basic paper due to time limitations on its oral presentation. In order to round out treatment of the subject, the deleted portion is being supplemented now, also schematically, to maintain the pattern of the original paper.

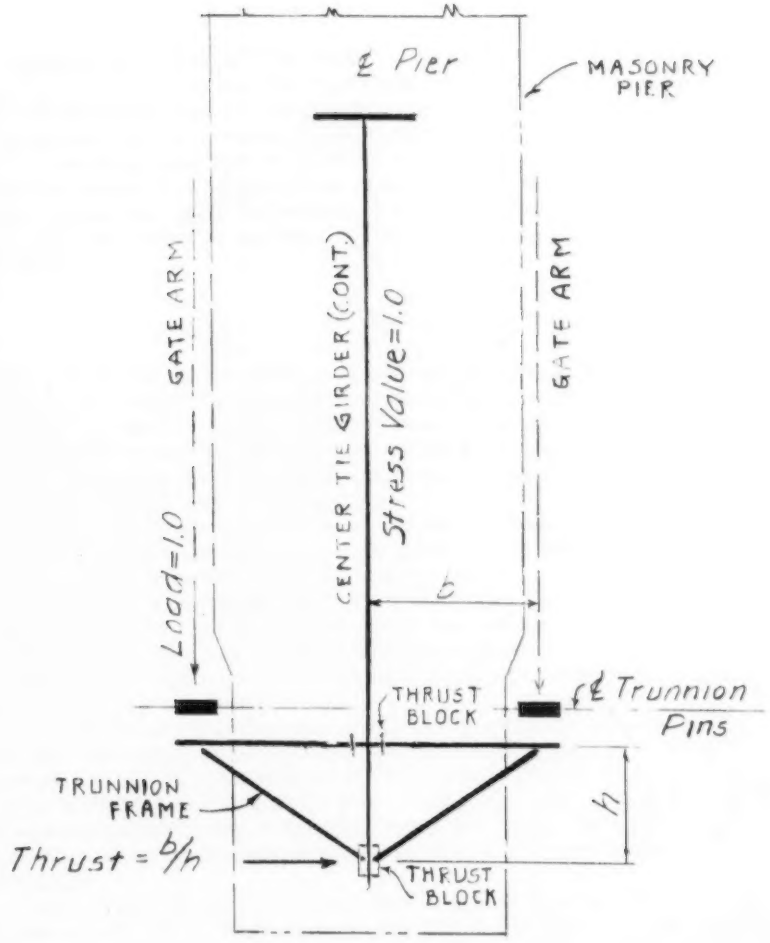
When overflow type tainter gates are employed, or when channel operating conditions dictate clear opening with the gates raised, their side arms are placed parallel and close as practicable to the faces of the spillway piers. Fig. 8 shows a skeleton arrangement of the anchorage when such gates are involved. It will be noted that the axially stressed triangular frame at the downstream end of the anchorage transmits only the actual hydrostatic thrust on the gates to the central tie girder in axial stress, similarly to the basic scheme of Fig. 5 of the paper, with the exception, however, that in this arrangement no flexural stresses are developed in the girder even under unequal loading on adjacent gates. When such loading conditions occur, the unbalanced horizontal forces are transmitted directly into the pier masonry by the two central thrust blocks indicated on the diagram, without affecting the purely tensile stress in the tie girder or its magnitude, which is merely the applied hydraulic thrust on the gates. In all other respects these anchorages are similar to the basic type of Fig. 5.

The writer is indebted to Mr. Wearne for his interest in the novel anchorages and for the opportunity to extend the fundamentals of the paper to parallel arms tainter gates.

FIG. 8

C-NEW ANCHORAGE

PARALLEL GATE ARMS



RIVERS UNDER INFLUENCE OF TERRESTRIAL ROTATION[a]

Closure by Otakar W. Kabelac

OTAKAR W. KABELAC,[1] M. ASCE.—Dr. Tison's elegant mathematical proof on dissimility of erosion phenomena in natural river channels (Proc. 1381-7) resulting from centrifugal force on concave banks and due to the terrestrial rotation and Coriolis effect, though seemingly in violation of syllogistic reasoning, may find its justification in the unrelated and fundamentally different origin of erosional forces involved. It makes doubtful the practicability of distorted hydraulic models, in which the Coriolis effect is substituted by a curve according to the formula (10) of the discussion and which gives for "R,"

$$R = \frac{V}{2 \cdot u \cdot \sin f}$$

where "u" is the angular velocity of terrestrial rotation and equal to 0.000073 Radians/sec, and "f," geographic latitude of the locations.

As to the use of rotational models, extensive studies are being carried out by SOCIETE HYDROTECHNIQUE DE FRANCE in connection with the discussion of Coriolis effect on the behavior of energy trajectories in tidal currents, which is needed for the planning of tidal powerplants along the CANAL LA MANCHE coast of France. The representation of tidal current energy phenomena on a fixed reduced scale model, failed to establish the circulation of the waves, generated by the rotation of the earth.

Mr. F. Biesel, INGENIEUR AU LABORATOIRE DAUPHINOIS D'HYDRAULIQUE, SOGREAH, calculated the average velocity of circulation in the Bay of SAINT-MALO as 1.2 kilometers/hour (0.648 knot) as result of Coriolis effect. He emphasized that this is only an average value, and that local velocities may be much higher.

The following remarks on the circulation phenomena in natural river channels, are intended as further contribution to the analysis, expressed mathematically, by the equations (4)-(9) of Proc. 1381-7, and also in addition to the original material in Proc. Paper 1208. The remarks are based on the frequently quoted theory of the hydrologist, N. S. Leliavski dealing with the formation of river channels, river regulation, sedimentation and related problems.

The circulation, the transversal motion in rivers, is generated by the Coriolis force due to the terrestrial rotation and also by the centrifugal force in a curved channel. (Fig. 1) The elementary theory on hydraulics of rivers teaches that flow velocity decreases with the river depth being maximum at

a. Proc. Paper 1208, April, 1957, by Otakar W. Kabelac.

1. Hydr. Engr., Corps of Engrs; U. S. Dept. of the Army, Washington, D. C.

the surface of the talweg and zero at the bottom, as represented by graph (Fig. 2a). The Coriolis force

$$P_{cor} = 2 \cdot m \cdot V \cdot u \cdot \sin f$$

and also the centrifugal force

$$P_{cen} = \frac{m \cdot V^2}{R}$$

both being the functions of longitudinal velocity "V," therefore vary consequently with the depth of the channel along the analogous graph (Fig. 2a), pertinent vectors directed towards the bank.

The excess hydraulic head created on the right and concave bank of the river, because of surface gradient and equal to tangent of the angle Y

$$\tan Y_{cor} = \frac{2Vu \sin f}{g} \quad \text{and} \quad \tan Y_{cen} = \frac{V^2}{Rg}$$

is constant along the vertical (Fig. 2b) with vectors directed from the right bank. The resulting vectors (Fig. 2a and 2b) are directed towards the right and concave bank in the upper layers as long as P_{cor} and P_{cen} are larger than the corresponding excess of hydraulic head. In lower layers, below the indicated limit, the vectors are in the opposite direction (Fig. 2c).

The resultant of the transversal and longitudinal velocities is the circulation (Fig. 3), indicated by the trajectories, deviating towards the right and concave bank in the upper layers, descending to the bottom and crossing the channel towards the convex and left bank, thus eroding the concave and right bank and carrying the sediment toward the convex and left bank.

All remarks are for rivers on the northern hemisphere, where the Coriolis force affects the right bank in straight as well as in curved reaches. The effect of centrifugal force increases the Coriolis effect when the concave bank is on the right one and decreases the Coriolis effect when on the left bank of the river, according to

$$P = P_{cor} \pm P_{cen} = mV \cdot \left(\frac{V}{R} \pm 2u \cdot \sin f\right)$$

As previously described the Coriolis force is apparent only on large rivers, particularly in tidal regions and under flood conditions.

The measurement of the circulation velocities performed by Soviet scientists for which LELIAVSKI designed instrumentation shows the following values for a "large river" (not specified) according to CHEBOTAREV.

Depth m	Longitudinal velocity m/sec	Transversal velocity m/sec
1.0	0.74	0.0394 towards right
3.10	0.73	0.0328 towards right
5.20	0.72	0.0136 towards right
7.30	0.64	0.0242 towards left
9.40	0.51	0.0616 towards left
10.30	-	-

The circulation in natural river channels affects greatly its formation, washing-out the right and concave banks, increasing progressively the number of bends thus ultimately forming meanders. The circulation and changes of its direction due to the alternation of bends, is also closely related to the formation of sandbars and shallow crossings, and to the motion of solids, particularly in cases of high velocities under flood conditions and low eroding resistance of the stream bed and banks. The Volga, Kura, and Amu-Darja are mentioned in Soviet sources as examples of rivers with a very heavy washout due to the circulation. On the Amu-Darja, the undermining effect on the right bank reaches catastrophic proportions each year, moving the banks inland as much as 100 m in 24 hours and involving large areas.

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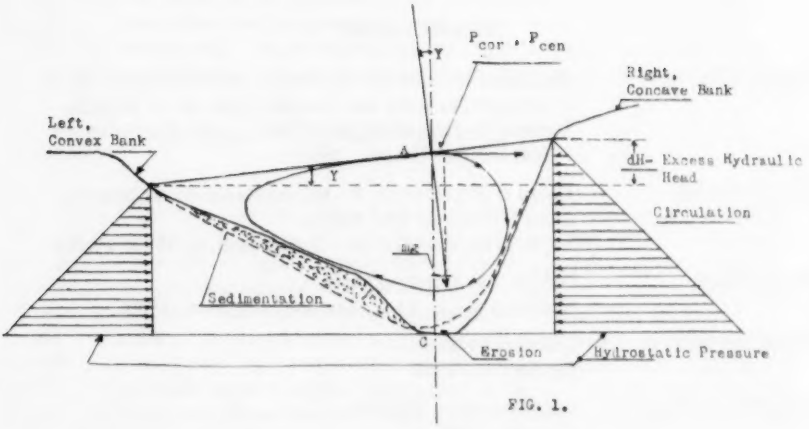
Y
P_{cor} , P_{cen}
Right,
Concave Bank
Left,
Convex Bank
A
dH- Excess Hydraulic
Head
Y
Circulation
Sedimentation
C
Erosion
Hydrostatic Pressure

FIG. 1.

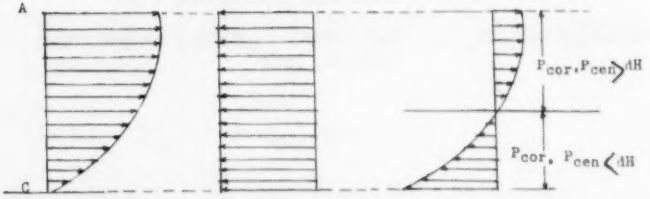
A
C
$P_{cor}, P_{cen} > dH$
$P_{cor}, P_{cen} < dH$
a.
b.
c.

FIG. 2.

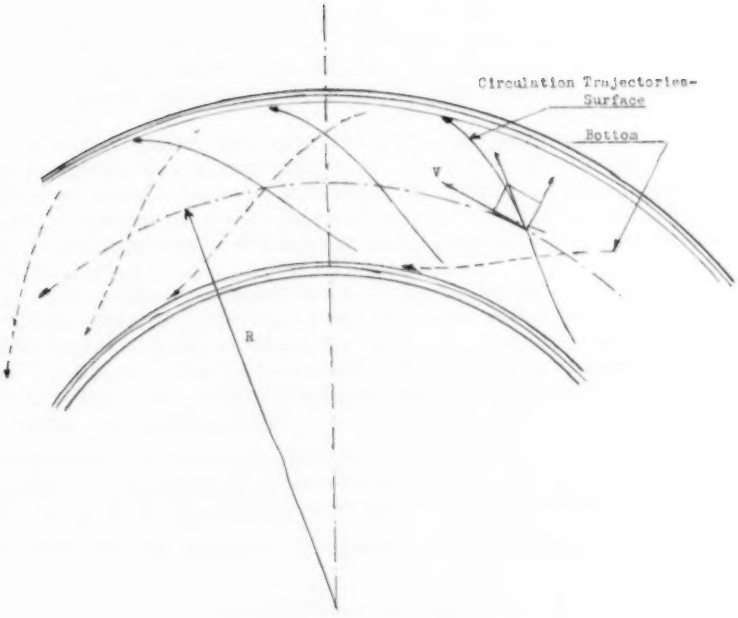

FIG.3.

THE LITTORAL DRIFT PROBLEM AT SHORELINE HARBOURS[a]

Discussion by R. Silvester

R. SILVESTER.[1]—The subject the author has chosen certainly warrants a lot of attention as the maintenance of harbour entrances is an expensive item in maritime administration. Suggestions might have been forthcoming, though, on improvements in design of the harbours cited or on possible lines of research.

As the author directs the reader to certain publications for a thorough discussion of sedimentation problems, it is only right that the reference list should be as complete and comprehensive as possible. It would appear that the first 20 references are for this purpose, all of which are of American origin; even though one or two authors are "foreign" the list does appear somewhat biassed.

The main conclusion reached by the author is that the novice should heed the warnings given by engineers of old. This, the writer submits, is rather a negative approach to the problem, where the greatest pitfall is to observe only half the picture. This may even have been done in the current article.

An up to date discussion of coastal sediment motion should include, among other things, such factors as:

a) The long and short term meteorological effects.
b) The various means by which sediment is transported.
c) The effect of wave spectra width on (b) and its generation by (a).

The writer will only outline some aspects of these phenomena.

a) Meteorological Conditions

Most waves, of any consequence in littoral-drift studies, are generated by winds in low pressure centres or cyclones. Anticyclones may be more persistent but the winds accompanying them are generally small in magnitude. Thus, if the location, duration and intensity of low pressure centres is studied, the direction and proximity of fetches (areas where waves are generated) can be determined.

This information leads to the direction of wave approach towards a particular section of coast and the types of wave to be expected from those directions. Taken over a year such a study may indicate the "balance of power" from upcoast and downcoast and when taken over a number of years a long-term trend may exhibit itself. Such a statistical analysis can assist

a. Proc. Paper 1211, April, 1957, by J. W. Johnson.

1. Senior Lecturer in Civ. Eng., (Hydraulics), Univ. of Western Australia, Nedlands, Western Australia.

in concurrent wave hindcasting or littoral drift observations.(1)

The writer has found, and believes it to be the rule rather than the exception, that storm waves approach a coast from a different direction from the predominent swell. This results in a reversal of littoral drift but, as pointed out in section (c) below, the mode of movement is different in each direction. It is complementary in that the storm waves assist the predominent swell to move greater quantities of sediment than if it works alone.

b) Mode of Movement

There are three main methods by which sediment is moved alongshore, namely

i) by suspension in the surf zone,
ii) by rolling in the surf zone,
iii) by saltation beyond the surf zone.

The author states: "Both model and prototype measurements indicate that most of the longshore transport of material occurs in the vicinity of the breakers where the available wave energy is converted suddenly from oscillatory motion into the form of turbulence." Of the references cited in this regard (author's 14 and 15), the first is an unqualified statement that 80% of the sediment movement takes place in the surf zone, whilst the second describes model tests along a supposedly infinitely long beach in which only the surf zone is studied anyway. In regard to the measurement of littoral drift in nature, the writer believes that no instruments have yet been devised to adequately cope with this problem.

When comparing the relative volumes of material moved by methods (i) and (ii) in the surf zone and by method (iii) beyond it, a balance must be struck between the apparent high velocities in the surf and the very large area of sea bed over which saltation actually takes place. Another factor not to be forgotten is that the finer sediment is sorted offshore and suffers saltation more easily than the coarser material near the beach. The term "saltation" was not used by the author although some authorities believe it to be the most important factor in littoral drift processes.(2,3,4)

The author states that "the exact mechanism of the movement around rocky headlands has not been completely established, but it probably is a combination of sand movement by turbulent action of waves breaking at the base of the precipitous cliffs, as well as by the movement of sand by wave action in depths to as much as 80 feet." The type of action is not suggested but it seems clear, from existing knowledge, that the mode of movement is saltation, together with a current near the ocean bed.

Saltation adjacent to a rocky headland is amplified by the standing waves or clapotis which are formed by the reflection of waves from the rock face. Experimental work has been carried out with clapotis(5) and the excessive bed movement near vertical cliffs has been observed.(6)

The current necessary to transport sediment continually thrown into suspension, across the ocean floor, can be due to three main causes—

i) that induced by the littoral current in the surf zone,
ii) ocean wide phenomena such as tides or wind drag,
iii) the mass transport of water due to the wave action.

When considered in relation to the sediment cycle discussed in section (c)

below, this last current (if it may be termed that) may become predominant.

The mechanism of saltation has been studied both mathematically and in two-dimensional models[7,8,9,10,11,12] and the complicated motion of the particules indicates the difficulty of measuring the net rate of movement. The writer feels that the author's table I is of little use since the sand collected behind a breakwater, such as that at Santa Barbara, is not the total movement of drift along the coast; probably half as much again actually bypasses the harbour.

By observing sediment movement with the use of heavy mineral tracers,[13,14] it has been shown that sediment does not always become deposited in quiescent water behind a breakwater or rocky headland, but is carried across the intervening bay by means of saltation and a current near to the ocean bed. In a recent investigation by the writer[15] aerial photographs were obtained which exhibited such a movement by a whitening of the water due to the "suspended" sand. Unfortunately the trace was not reproducable in the published article.

c) Wave Spectra

As mentioned in section (a) two main types of wave may approach a coast; (i) storm waves which are still being generated when they reach the coast (i.e. the fetch is adjacent to it), and (ii) swell which has been generated some distance from the coast and has travelled through an area of little or no wind (i.e. an area of decay).

i) Storm waves have certain characteristics—they are made up of waves of many periods, that is the wave spectrum is wide[16] (the maximum period present is determined by the length of the fetch)—they are multi-directional since a wind generates waves at angles up to 30^{o} to its general direction—they are peculiar in shape, being steep in front and mildly sloped at the back and they are steeper than swell waves.

Due to the large amount of water thrown towards a coast by storm waves, it quickly becomes eroded and material is moved from the beach to form a bar offshore, which, once established, prevents subsequent waves from attacking the beach, forcing them to break offshore. It is felt that the figure of 0.025, quoted by the author as the critical steepness in this regard, is of little practical significance when conditions in nature are considered.

The short period waves present, which are not refracted so much as the longer period waves, generally break at a greater angle to the beach and hence tend to generate a stronger littoral current. This is more than offset by the multi-directional nature of their approach and the resultant motion is a slow current over a wide band of water (from the bar to the beach) in which a large amount of sand is suspended and rolled. Beyond the bar or breaker zone saltation is taking place and again, due to the multi-directional nature of the wave movement, stable ripples are not formed and a greater amount of material than normal can be moved alongshore by the induced current from the surf zone.

ii) Swell consists of waves that have emanated from a fetch and are in the process of spreading laterally as well as in the direction of propagation. Thus at some distance from a fetch waves occur which are more unidirectional and have a small band of periods. The effect of this narrowing of

the wave spectrum on the mass transport of water at the ocean bed has been studied mathematically.(17)

The saltation and mass transport actions of swell permit it to sweep sand towards the shore and here the assistance of the infrequent storm waves is seen as they move material offshore for the swell to operate on. In the process of moving shorewards the sand is moved alongshore if the swell approaches the coast at some angle, which is generally the case.

Although the persistent swell determines the direction of the net littoral drift it is the intermittent occurrence of storm waves, together with saltation offshore, that permits groynes and breakwaters to be bypassed by sand. The longer the waves the wider is the band of ocean affected by saltation, whereas, in the case of 7 second waves and less, the bed outside the surf zone is hardly "touched." Thus, if models are to represent a prototype completely, storm cycles should be included in tests and sedimentary material should be used which can sort itself in a similar manner as in the prototype. These requirements present many difficulties.

Other points in the text not already mentioned are as follows:

a) The writer sees no reason in figure 4 for accretion to take place in the dredged channel above the general profile of the adjoining beach. Whether this is implied is not clear, but it is felt that nature will bridge the gap only up to the level of the existing upshore beach and the breaker line will not be deflected seawards as shown.

b) In regard to figure 5, there are few breakwaters where waves are completely dissipated by breaking. The formation of a beach along the ocean side of such a breakwater will depend upon

i) the reflective power of the breakwater,
ii) the rate of sand supply from upcoast,
iii) the tidal range.

The impression might be gained that where there is little or no breaking there is no sediment movement, which is quite erroneous. There may even be more.

Although the author advises the reader to observe littoral processes at adjoining littoral barriers in order to determine the existence or non-existence of drift reversals, he includes no such probabilities in the figure. Reversal is so frequent that a warning should be issued to engineers that, in such a harbour as depicted in figure 5, accretion is very likely within the body of the harbour. Material will be moved from the downcoast area to sheltered regions within the harbour. This also aggravates the erosion problem. Again, the bi-directional approach of waves will force the accretion at the tip of the breakwater along its inside face and so make inoperative any wharves or jetties located there. Figure 22 should be compared in this regard.

With reference to the dredging of material behind a breakwater tip, where reversals of drift may occur, the time of the year must be chosen with care to prevent material deposited downcoast being forced into the harbour by the spasmodic storm waves. (A recent example in which weather conditions were ignored in dredging operations is in the sand bypassing of Port Hueneme.)(18) The writer cannot envisage the necessity of bypassing material in both directions at different times of the year; almost invariably a drift in one direction will predominate.

c) Figure 7 depicts an offshore breakwater and its effect on a coastline. Although the apex of the resultant accretion will vary with the direction of the predominant waves, it would seem that the one shown is a little too far upcoast, especially when compared with figure 21 and others occurring in reference (19), cited by the author.

The length and disposition of an offshore breakwater determines its effect on the adjacent shoreline. It may be so short or so far from shore that "the littoral material will move along the coast uninterrupted by the presence of the structure and consequently no maintenance problems from sediment deposition is involved." Generally the structure is not carrying out its intended purpose in these circumstances.

Mention could have been made of the efficacy of several offshore breakwaters in line spaced echelon fashion as practised successfully many years ago at Cochin Harbour.(19) The diffraction of the waves through the gaps is sufficient to cause deposition with an accompanying saving in material.

d) In the references the designation of page numbers in which particular subjects are mentioned, rather than the commencing page, is a change in tradition which is not particularly helpful.

The words of the Chairman (Dr. G. E. R. Deacon) to the Commonwealth Oceanographic Conference held at the National Institute of Oceanography, England, in 1954, are very pertinent to this discussion. It is reported:(20) "The Chairman said that very little precise knowledge of wave characteristics was brought to bear on the design of ships, harbours and coast defences. Enormous sums of money were spent on engineering projects which might well be carried out more effectively if a fraction of the cost were devoted to increasing understanding of the underlying physical processes in the sea. It seemed to him that coastal engineering was a very vague subject, in which were any number of schools of thought."

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)c, 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238 (WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253 (IR1), 1254(ST3), 1255(ST2), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296 (HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304 (ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312 (ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320 (ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328 (AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337 (SA4), 1338(SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345 (HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST6), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374 (HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(ED2), 1492(HY6), 1493(BD2).

VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502(IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520(EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

c. Discussion of several papers, grouped by Divisions.

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